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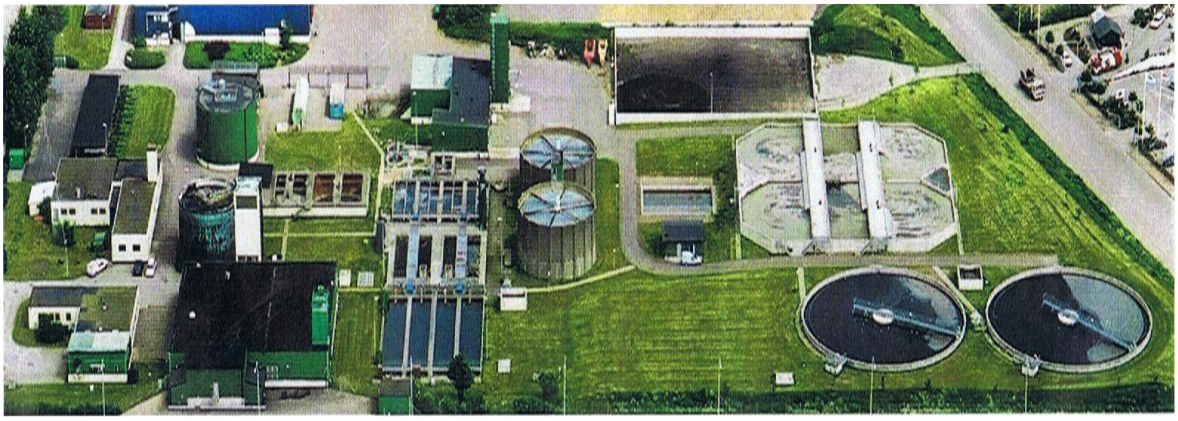
Water and Environmental Engineering
Department of Chemical Engineering

Troubleshooting for improved bio P at Lundåkraverket wastewater treatment plant, Landskrona, Sweden



Master's Thesis by

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Picture on front page:

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Aerial photograph of Lundåkraverket WWTP
(Courtesy:Jan-Erik Petersson, Lundåkraverket, Landskrona)

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Abstract

Phosphorus is said to be one of the key nutrients responsible for eutrophication. Therefore it is very important to remove phosphorus before being discharged into the receiving waters. In wastewater, sources of phosphorus are mainly from washing detergents and human wastes. Enhanced biological phosphorus removal (EBPR), also popularly known as the bio P process is a biological process that removes phosphorus without the use of chemicals. In this process, it is important to create suitable environments for the presence and growth of a special type of bacteria called the Phosphorus Accumulating Organisms (PAOs) which are capable of storing large amounts of polyphosphates in their cells. Lundåkraverket WWTP is situated at the municipality of Landskrona, Sweden. Occasional phosphorus peaks in the effluent have been observed in this WWTP with P concentrations exceeding the maximum permissible limits. These peaks normally last for a week and after that they disappear. The time periods between the occurrence of these peaks is normally two to three weeks and sometimes even less. It is a matter of concern to the concerned authorities at Lundåkraverket WWTP. Therefore, an attempt was made to simulate the WWTP process conditions in order to study the P peaks. Computer model EFOR was used as a tool for the simulations. Only the biological part was modelled and the chemical part was completely ignored. A constant wastewater flow of 700 m3/h and yearly average values were taken as inputs for the wastewater composition. A constant temperature of 20°C was used throughout. Factors affecting the bio P process such as diluted wastewater concentration, low-flow wastewater, temperature effects, high acetate wastewater concentration and operational problem related to return sludge pump were studied. The results indicated that the effects of a diluted wastewater composition on the P peaks were noticeable with effluent P concentrations as high as 3.2 mg/l which was eight times higher than the normal P concentration of 0.39 mg/l. Low-flow conditions in the wastewater composition was maintained by decreasing the constant wastewater flow from 700 m3/hr to 300 m3/hr. No obvious changes in the effluent P concentrations were observed. Results showed slightly high P concentrations that were within the allowed P effluent limits at Lundåkraverket. Similarly temperature and high acetate dosing also didn't show any visible P peaks. Increase in acetate concentration lowered the P concentrations from 0.39 mg/l to 0.17 mg/L. Lundåkraverket WWTP was also experiencing problems with filamentous organisms mainly *Microthrix parvicella* which was causing foaming problems in the settlers. A detailed literature study was performed to find possible solutions for the problem. A number of alternatives have been presented in the literature which includes reduction of SRT, maintenance of DO concentration, pre-treatment by floatation, chlorination and dosage of PAX-14.

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Preeti Rajbhandari Shrestha & Sujay Shrestha

Nomenclatures

ATP	Adenosine Tri-Phosphate
ATS	Aeration Tank Settling
BNR	Biological Nutrient Removal
BOD	Biochemical Oxygen Demand
BPR	Biological Phosphorus Removal
C	Carbon
cBOD	Carbonaceous Biochemical Oxygen Demand
CLSC	Confocal Laser Scanning Microscope
COD	Chemical Oxygen Demand
DN	Denitrification
DO	Dissolved Oxygen
DWF	Dry Weather Flow
EBPR	Enhanced Biological Phosphorus Removal
FIA	Flow Injection Analysis
FISH	Fluorescence in-Situ Hybridisation
GAOs	Glycogen Accumulating non poly-P Organisms
LOI	Loss of Ignition
MAR	Micro Auto-Radiography
MLSS	Mixed Liquor Suspended Solids
N*	Nitrification
N	Nitrogen
P	Phosphorus
PAOs	Phosphorus Accumulating Organisms
PHB	Poly-hydroxy-butyrate
RAS	Return Activated Sludge
RWF	Rain Water Flow
SCADA	Supervisory Control and Data Acquisition
SRT	Sludge Retention Time
SS	Suspended Solids
SVI	Sludge Volume Index
TF	Trickling Filter
TOC	Total Organic Carbon
VFA	Volatile Fatty Acids
WWTP	Wastewater Treatment Plant

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1.0 Introduction

1.1 Background

Wastewater handling and treatment is important in the society to prevent the adverse affects of untreated wastewater for human health and ecosystem. The presence of nutrients, specially phosphorus and nitrogen in wastewater can cause serious problems in the receiving water bodies such as eutrophication which is a common term used to indicate algal blooming in natural water. This kind of algal blooming causes dissolved oxygen depletion which is harmful for the living organisms in the water bodies. Nitrogen is present in wastewater partly as organic nitrogen and partly as inorganic nitrogen in the form of ammonium, nitrite and nitrate. Nitrogen oxidising bacteria use a significant amount of oxygen to convert ammonium to nitrate through the process nitrification thereby causing oxygen depletion. Phosphorus on the other hand is found in wastewater as organic phosphorus as well as inorganic phosphorus in the form of polyphosphate and orthophosphate. Principle sources of phosphorus are mainly phosphate detergents and human wastes (Gillberg et al., 2003). Owing to these negative consequences it is important to limit the concentration of these nutrients, mainly nitrogen and phosphorus below a certain level. Different countries in the world have their own standards for the maximum permissible value of effluent phosphorus and nitrogen concentrations from WWTP to receiving waters which has to be strictly followed. This limiting value for effluent phosphorus concentrations in Sweden is typically very low.

Phosphorus in wastewater can be removed by treating it either chemically or biologically or combination of both, in a wastewater treatment plant depending on the wastewater characteristics and also on the accepted effluent standards. In enhanced biological phosphorus removal process (EBPR), a special type of bacteria, the bio P bacteria or in other words phosphate accumulating organisms (PAOs) is responsible for storing large quantities of soluble orthophosphate in the form of insoluble polyphosphate in their cells (Janssen *et al*., 2002). During the anaerobic phase of wastewater treatment, the PAOs take up carbon sources such as acetates or VFAs present in the wastewater and store them as carbon-rich product such as poly-hydroxy-butyrate (PHB). This energy is obtained mainly by the breaking up of the stored polyphosphates which ultimately releases orthophosphates in the water phase. In the subsequent aerobic/anoxic phase the PAOs use the stored PHB as energy source for P uptake, polyphosphate storage and biomass growth. Therefore, phosphate is removed from the wastewater with the help of excess sludge (Janssen *et al*., 2002).

Lundåkraverket is a modern WWTP situated in the municipality of Landskrona, which lies in the south-most part of Sweden, also known as the Skåne. Ever since its operation in 1948, it has undergone a number of technical changes. After the late 90s, occasional phosphorus peaks in the treated effluent were measured. This problem of occasional sporadic high P concentrations in the outlet still persists and has been a matter of concern for the local management.

1.2 Aim

The main objective of carrying out this study is to thoroughly understand the EBPR process in Lundåkraverket WWTP, Landskrona, and also to carry out a study of the ongoing occasional abnormal phosphorus peaks in the outlet with the help of computer modelling. An

attempt to identify the possible reasons for the phosphorus peaks by simulating the actual process conditions in the computer model EFOR will also be done.

Operational problem such as bulking and foaming mainly due to the presence of *Microthrix parvicella* have been observed at Lundåkraverket WWTP. Finding probable reasons and solutions with the help of literature reviews for this particular type of problem is also a part of the study.

1.3 Limitations

An approach to analyse the problem associated with the occasional high peaks of phosphorus in the effluent from Lundåkraverket WWTP has been done with the help of available data and information, literature reviews and computer modelling.

DHI software EFOR has been used for computer modelling. Input data for incoming wastewater characteristics, dimensional data for the different components of the WWTP is based entirely on available data. Annual average values from the available data have been used in the model while daily load variations and storm water effects have been neglected. This may also have caused some inaccuracy in the model results. Effluent concentrations from the individual treatment units particularly, the primary and secondary settlers are not available for accurate calibrations in one of the models. Lack of appropriate data of incoming wastewater flow and the effluent parameters from trickling filters are also another constraint that has limited the study.

Computer modelling is strictly limited only for the biological treatment. Chemical treatment, which is also the final treatment in Lundåkraverket WWTP, has been excluded. Therefore, elimination of this process in the modelling may limit the accuracy of the results. As a major portion of the wastewater treatment is based on EBPR, main focus of study is based on biological treatment.

2.0 Forms and sources of contaminants in wastewater

Water is vital for survival. Every human being needs water for their survival and their household works such as drinking, bathing, flushing toilets, dish washing as well as industrial use, agricultural use etc. Human consume clean water and after being consumed gets contaminated and turns into wastewater which possess a number of substances along with it. Wastewater contains most of the substances that is found in society which can be divided into suspended solids, oxygen-demanding substances, nutrient salts, bacteria, viruses, parasite spores, heavy metals, environmentally harmful substances. These contaminants can be classified by dividing them on the basis of particle size or dividing them into organic and inorganic substances.

According to particle size the particles less than 0.1 µm are considered as dissolved particles, size from 0.1-1.0 µm are colloidal, size from 1 -100 µm are suspended and size greater than 100 µm are referred to as sedimentary suspended particles (Gillberg *et al*., 2003).

Generally, the organic contaminant consists of one third of dissolved, colloidal and suspended substances. The inorganic material comprises mainly the dissolved substances.
The organic substances in municipal wastewater as determined by HYPRO project shows the breakdown of organic substance as shown in Table 1 below (Gillberg *et al*., 2003).

Table 1: Organic constituents of municipal wastewater (Gillberg *et al*., 2003)

Substance	**Percentage of organic carbon in wastewater**
Carbohydrates	11-18%
Proteins	8-10%
Free amino acids	0.5-1.5%
Higher fatty acids	23-25%
Soluble organic acids	7-11%
Esterified fatty acids (fat)	9-12%
Surfactants	4-6%
Others	25-28%

The concentration of organic substances is measure as biochemical oxygen demand (BOD), chemical oxygen demand (COD), loss of ignition (LOI) and total organic carbon (TOC) (Gillberg *et al*, 2003).

Biochemical oxygen demand (BOD) is defined as the measure of biodegradable substances in the wastewater. Oxygen will be consumed while breaking down these substances by bacteria. So in order to measure the oxygen demand, the measurement of oxygen used by the microorganisms over a period of 5 days (BOD_5) or 7 days (BOD_7) to breakdown the organic contaminant at a temperature of 20°C is done. A measurement of BOD is done in units of mg oxygen/l or g oxygen/m^3 (Gillberg *et al*, 2003).

Chemical oxygen demand (COD) is the measure of the concentration of contaminants in the wastewater that can be oxidized by chemical oxidizing agent such as potassium dichromate and potassium permanganate at high temperature. The organic content is measured by the amount of oxidizing agent required and it is converted to equivalent oxygen concentration, so it can be measured in units of mg oxygen/l or g oxygen/m^3 (Gillberg *et al*, 2003).

Loss of ignition (LOI) is defined as the change in percent weight in a dry substance when the sample is ignited, i.e. heated to 550°C according to Swedish standard (Gillberg *et al*, 2003).

Total organic carbon (TOC) is the amount of organic matter which is measured by amount of carbon dioxide produced when sample is burned. It is measured in unit of mg C/l (Gillberg *et al*., 2003).

Wastewater consists of many harmful materials and nutrients that may cause problems in receiving water if discharged without treatments. Phosphorus and nitrogen is a nutrient which if discharged without lowering its limits in receiving water will cause severe problems like eutrophication. It is for this reason, wastewaters must be treated before it is discharged into the water bodies. In municipal wastewaters the sources of phosphorus are primarily washing detergents and human wastes (Tykesson, 2002).

The limit for discharging total phosphorus is very low for Sweden and most of the treatment plants have maximum limits of 0.3 or 0.5 mg/l total phosphorus in effluent as monthly or quarterly mean value, depending on the sensitivity of recipient water (Tykesson, 2005).

So in order to remove the contaminants such as phosphorus, nitrogen and other harmful substances, the wastewater undergoes various treatment steps in wastewater treatment plants (WWTPs). There are different configurations of treatment plants available of which few are mainly focused for removal of phosphorus and nitrogen. Also, different problems are encountered in treatment plants such as occasional high phosphorus peaks, bulking and foaming problems and sometimes unwanted phosphorus release in settlers etc. In order to find out a solution for all these associated problems, a clear understanding of wastewater treatment steps and the process involved is necessary. Therefore, a theory on a typical wastewater treatment process in WWTP is presented in the following section.

3.0 Wastewater treatment basics

Municipal wastewater is pumped up to the wastewater treatment plant through the pump station. Then the incoming wastewater undergoes various treatment processes in the WWTP. These treatment processes are described below.

3.1 Preliminary treatment

Preliminary treatment of incoming wastewater is done to remove large pieces of material such as fibrous debris, grit and other solid materials. Preliminary treatment helps to prevent physical damage to equipment, especially pumps and bearings, in the downstream treatment processes. Preliminary treatment includes screening, grit removal/sand trap etc (Brett *et al*., 1997).

3.1.1 Screening

Screening is the first treatment process of incoming wastewater where fibrous material and large pieces of solid materials such as rags, sticks, and plastics are removed. (Brett *et al*., 1997). Bar screen consists of a series of parallel bars or a perforated screen placed in a channel. Coarse screen have 6 mm and larger openings whereas fine screen have openings of 1.5 to 6 mm (EPA, 2003). Coarse solids are trapped on bars when flow passes through the screen. The screen should be manually or mechanically cleaned in order to prevent blockage of flow (Spellman *et al*., 2003).

3.1.2 Grit removal / Sand trap

Grit removal is done by using grit/sand traps, or chambers in order to avoid damage to the downstream pumps bearings and seals. In the sand traps heavier particles such as sand or soil are allowed to settle out by gravity in an area of lower velocity (typically 0.3 m/s). But the lighter organic materials are remained in suspension. The settled materials are removed from the bottom of the sand trap/grit trap by a submerged scraper and disposed off, normally to land fill (Brett *et al*., 1997). It is also common to include some aeration in the sand trap in order to keep the water oxygenated and in order to improve grease removal (Gillberg *et al*., 2003).

3.2 Primary treatment

3.2.1 Primary sedimentation tank

Primary settling tanks are those which receive wastewater before it will undergo biological treatment. Settling tanks may be rectangular or circular in shape where water is allowed to remain quiescent in order to settle out particulate solids in suspension**.**

Influent from the preliminary treatment process enters the primary sedimentation tanks where organic and inorganic materials are removed by settling. Velocity is comparatively low in the primary sedimentation tanks due to the relative size and position of the exit from the tank (Brett *et al*., 1997). Settleable solids are removed due to the low velocity and retention times in the tank which is in the range of 1.5 to 2 hours. Settleable solids also consist of BOD; therefore by removing settleable solids will also remove about 25% to 35% of BOD. Thus, in these primary sedimentation tanks, about 90-95% of settleable solids, 50-70% of suspended solid and 10-15% of total solids will be removed. (Haller, 1995)

Rectangular tanks may have length to width ratio of about 3:1 to 5:1 with liquid depths of 2 to 2.5 m. The bottom of the tank has a gentle slope toward the sludge hopper (Hammer, 1986). The organic material (primary sludge) collected at the bottom of the tank is removed by a scraper. The primary sludge is pumped to the sludge facility. The scum baffle will prevent the fat, grease and other floating materials from entering the downstream process. The material collected by the scraper is removed periodically to maintain the continued effectiveness (Gillberg *et al*., 2003).

3.3 Secondary treatment

The effluent from the primary treatment will pass to secondary treatment process where a large portion of the remaining BOD, COD and SS will be removed by the biological action. This treatment process consists of aerobic (presence of oxygen), anaerobic (absence of oxygen) and anoxic (presence of nitrate and absence of oxygen) processes. Majority of treatment plants within EU uses the secondary treatment by trickling filters, activated sludge plants or modification of these two processes. Generally secondary treatment methods are classified depending on their aerobic and anaerobic nature. (Gillberg *et al*., 2003)

Different types of secondary wastewater treatment configurations are available. Here only the trickling filters and the biodenipho process are discussed in the preceding sections. The reason for describing only these two methods is to understand the similar methods that are used in Lundåkraverket WWTP.

3.3.1 Trickling filters

A trickling filter (TF) consists of a bed of highly permeable media such as rocks, slags etc. As wastewater passes through the filter media, a slime layer of biological film of microorganisms (aerobic, anaerobic and facultative bacteria, fungi, algae and protozoa) are formed in the filter media. Organic materials in the wastewater are adsorbed by the microorganisms attached to the filter media. These organic materials are degraded by the aerobic microorganisms. So it is essential to provide sufficient air for the successful operation of the trickling filters (Solomon *et al*., 1996). As more and more wastewater flows through the trickling filters the more the slime layers thicken (with microbial growth) and entry of oxygen is restricted to the media face. The increase of the thickness of slime layer, causes sloughing of slime. There are two general types of trickling filters configuration: single stage and two (or separate) stage. In single stage configuration of trickling filter, the removal of organic carbon or carbonaceous BOD (cBOD) occurs in a single unit. In Two stage configuration of trickling filter, carbonaceous BOD is removed in first treatment stage and nitrification occurs in the second stage (Solomon *et al*., 1996).

Performance of nitrification process is dependent on various factors such as, the availability of oxygen (i.e., adequate ventilation), ammonium nitrogen concentration, cBOD level, media type and configuration, hydraulics of the TF, temperature and pH etc.

In single-stage TF, for achieving adequate nitrification the organic volumetric loading should be within approximate ranges such as, if the filter media is of rock, 75% to 85% of nitrification can be achieved for the loading rate of the range 160 to 96 g BOD/m^3/d whereas 85% to 95 % of nitrification can be achieved for the loading rate of 96 to 48 g BOD/m^3/d. Similarly, for a plastic filter media, nitrification of 75% to 85% can be achieved for the loading rate of range 288 to 192 g BOD/m^3/d (Solomon *et al*., 1996). The plastic filter media can achieve the same degree of nitrification for higher organic loading because of its greater

surface contact area per unit volume than rock or slag. Usually TFs are designed with minimum effluent recycling capabilities in order to maintain stable hydraulic loading during normal seasonal operations. But by increasing the recirculation ratio and air circulation, the concentration of dissolved oxygen (DO) and thus nitrification is also increased. As DO is an important factor, sufficient ventilation should be maintained for proper operation of TFs. For high carbonaceous loading conditions, the effects of pH and temperature can be often avoided if proper DO concentration is maintained (Solomon *et al*., 1996).

3.3.2 Biodenipho configuration for secondary treatment of wastewater

There are different configurations of WWTP available for the biological nutrient removal (BNR) for nutrients such as phosphorus and nitrogen and also organic matter (BOD) and suspended solids (SS). Enhanced biological phosphorus removal (EBPR) can be achieved by introducing an anaerobic phase ahead of the aerobic phase in the existing treatment plants (Baetens, 2000). Here, an alternating biodenipho process configuration of WWTP has been explained.

The alternating biodenipho process was developed by I. Krüger Systems in cooperation with the Department of Environmental Engineering at the Technical University of Denmark. This process is mainly designed for the biological nitrogen and phosphorus removal (Isaacs *et al*., 1994). The biodenipho process is a modified form of biodenitro process in which an anaerobic tank is placed in front of the aeration tanks in order to enhance the bio P process as shown in Figure 1.

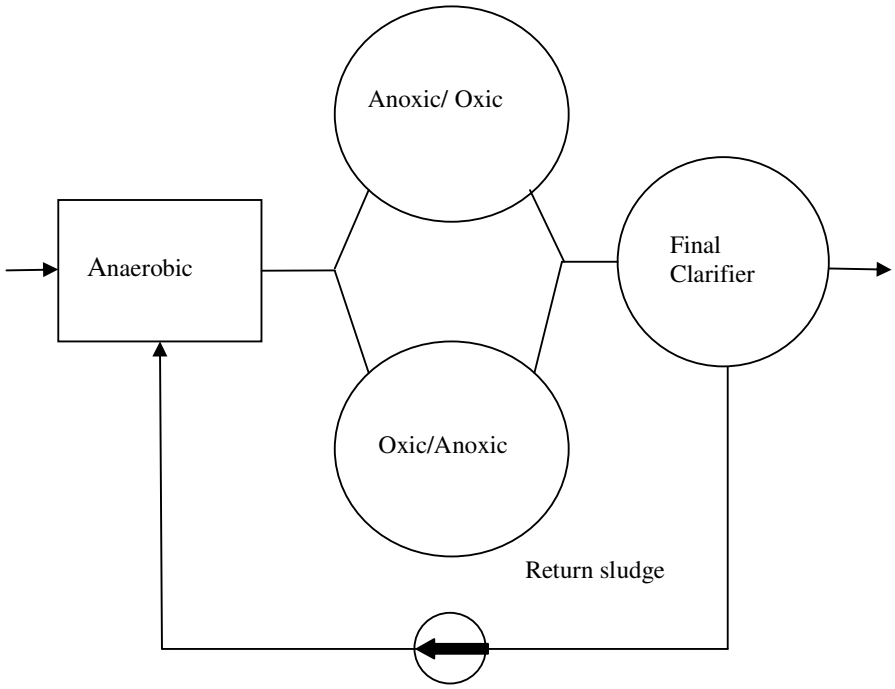

Figure 1: Schematic diagram of biodenipho process

3.3.2.1 Mechanism

The biodenipho process is based on biodenitro process. Biodenitro is an alternating process where nitrification and denitrification occurs alternately in two coupled aeration tanks (oxidation ditch). Wastewater flows in a changing sequence in these two tanks which are used in pairs using a submerged connection and are controlled by influent and effluent

valves. These tanks consist of aeration and mixing equipments and are alternately aerated and fed with wastewater (Janssen *et al*., 2002). Brush aerators known as rotors are used in both the tanks for maintaining oxic condition during nitrification phase. Most of the time one reactor (tank) is aerated while the other remains anoxic. Wastewater can be fed in either of the two tanks and effluent can be withdrawn from each of the tanks and flow can occur in either direction in between the two tanks. Biodenitro process is aimed to operate two reactors (tanks) in counter phase (equal loading). Usually the influent is fed into the anoxic reactor for the purpose of utilizing the organic carbon for denitrification. If the ammonium concentration reached close to zero before nitrate concentration in anoxic reactor then aeration can be switched off in the nitrifying reactor (Lukasse *et al*., 1999).

Denitrification occurs in the non-aerated phase (anoxic condition). In the biodenipho process there is a separate anaerobic tank which is compartmentalized to simulate plug flow characteristics (Janssen *et al*., 2002). The anaerobic zone is fed with mixture of wastewater and the return activated sludge (RAS) from the secondary settler (sedimentation tank/clarifier). The flow rates of the inlet water and return sludge are controlled by two separate pumps (Isaacs *et al*., 1994). The anaerobic tank has at least a retention time of one hour. Part of the influent bypasses the anaerobic basin at a high rain weather flow (RWF): dry weather flow (DWF) ratio. The operations with sequencing phases are the same in both biodenipho and biodenitro process. The duration of each phase varies, but it usually remains between 0.5 to 1.5 hours (Janssen *et al*., 2002).

The effluent from the anaerobic tank will be taken to non aerated zone (anoxic zone) where the remaining organic carbon in the wastewater will be utilized for denitrification process. At the same time anoxic phosphorus removal will also take place (Janssen *et al*., 2002). The bio solids are allowed to settle in the secondary settlers and hence the solids are removed or separated from wastewater (Princeton Indiana, 2004).

3.3.2.2 Process Description

In the biodenipho process, the mixed liquor (influent) passes through the oxidation ditch (tank) in series most of periods of time, or phases, whereas there are phases in which the influent passes only through one ditch while the other ditch remains isolated. Initially nitrification and denitrification volumes are determined depending on the composition of influent wastewater. And on this basis, the initial sizing of phased isolation is done in which, a sufficient aerobic solid retention time is also incorporated in order to remove carbonaceous BOD and to ensure complete nitrification at lowest anticipated wastewater temperature. A safety factor is also applied in order to accommodate for the rate and volume of flow for diurnal and seasonal variations. The anoxic and oxic volumes for nitrification and denitrification of wastewater can be provided by varying the duration of the phases (Princeton Indiana, 2004)

A biodenipho process of one complete cycle of operation of 4 hours consisting of four separate phases through the removal stages of nitrogen and phosphorus will illustrate well and help to understand the theory and operation of this process. One complete cycle of four phases labelled as B, E, G and J are considered below, where phases G and J are the “mirror image” of phases B and E respectively (Princeton Indiana, 2004).

In every phase, the influent wastewater is mixed with the Return Activated Sludge (RAS) before directing to either ditch. There is an influent and effluent weir located in the ditch. Influent and effluent distributors are motor operated weirs and are a part of Kruger Ditch

system design which is connected to the Kruger SCADA system. The SCADA (Supervisory Control and Data Acquisition) system is mainly a graphical representation of the physical plant used to monitor and control the plant and its equipment (Princeton Indiana, 2004). It shows all the monitored components such as the pumps, valves and other monitoring equipments. These weirs are raised or lowered (opened or closed) as needed during the plant operating cycle (Princeton Indiana, 2004).

Phase B

The cycle starts with phase B. In this example, phase B has a duration of 90 minutes. Effluent weir in ditch 1 is raised and effluent weir in ditch 2 is lowered, so that the hydraulic gradient is shifted and the flow direction is from ditch 1 to ditch 2 and finally the effluent to the final clarifier is discharged from ditch 2 (see Figure 2).

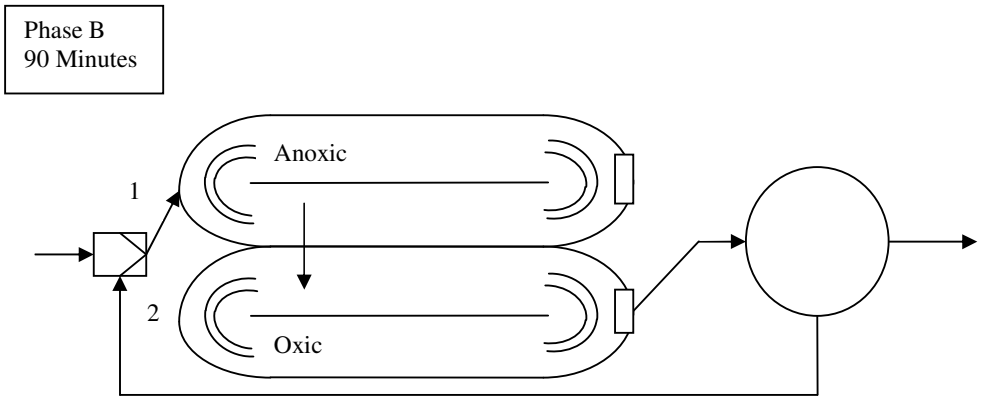

Figure 2: Ditch 1 - denitrification (anoxic), Ditch 2 - nitrification (oxic)

Ditch 1 and ditch 2 are operating in denitrification and nitrification modes respectively. During phase B, the rotors in ditch 1 are turned off for creating anoxic conditions, whereas the activated sludge in ditch 1 remains in suspension by submerged mixers. As there is an anoxic condition in ditch 1, the ammonium concentration in ditch 1 will be rising due to the ammonium and organic material in the influent. Influent wastewater is directed to the ditch 1 (anoxic state), so the carbon source required for denitrification will be provided by the influent wastewater. The accumulated nitrate in the previous cycle in ditch 1 during oxic phase (phase J), will now be transformed into free nitrogen by means of organic matter (BOD) due to denitrification in this ditch. Hence the concentration of nitrate is decreasing in ditch 1 in phase B.

In ditch 2, the ammonium concentration will decrease whereas the nitrate concentration will rise due to the oxic (aerobic) environment.

By lowering the motorized effluent weir in Ditch 2, the activated sludge in ditch 2 will be carried to the clarifier (secondary settler). The effluent will always be discharged from an oxic ditch to ensure that influent ammonium will go through nitrification period before exiting the ditches, so that concentration of ammonium in effluent is minimized.

Phase E

In phase E (See Figure 3), ditch 1 is isolated from the influent. The rotors in ditch 1 are turned on to produce oxic conditions. The increased ammonium concentrations in Ditch 1 from phase B will now decrease as a result of nitrification. Phase E, in this example has a duration of 30 minutes and both the ditches will be kept in oxic condition during this period. The influent flow will be directed to ditch 2 now instead of ditch 1, by switching the influent weir in the distribution chamber towards inlet for ditch 2. Mainly the distribution chamber is operated automatically via the PLC (Programmable Logic Computer), but during the event of emergency the unit can also be operated manually. Ditch 2 which was oxic from the previous phase will remain oxic and also the effluent will discharge from the same ditch 2 throughout this phase.

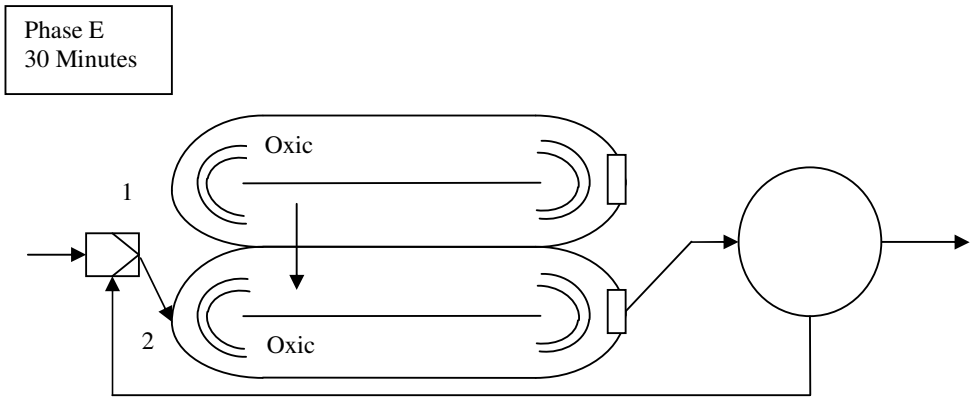

Figure 3: Ditch 1 – nitrification (oxic), Ditch 2 – nitrification (oxic)

Phase G

In this example, phase G has duration of 90 minutes and is a mirror image of phase B. Effluent weir in Ditch 2 is raised and effluent weir in Ditch 1 is lowered. So, due to the change in hydraulic gradient, the direction of flow will be from Ditch 2 to Ditch 1 and finally on to the clarifiers. During this phase, the rotors in Ditch 2 are turned off to create anoxic condition and the entire nitrates that are produced during the three previous oxic phases will be denitrified. Please refer to Figure 4.

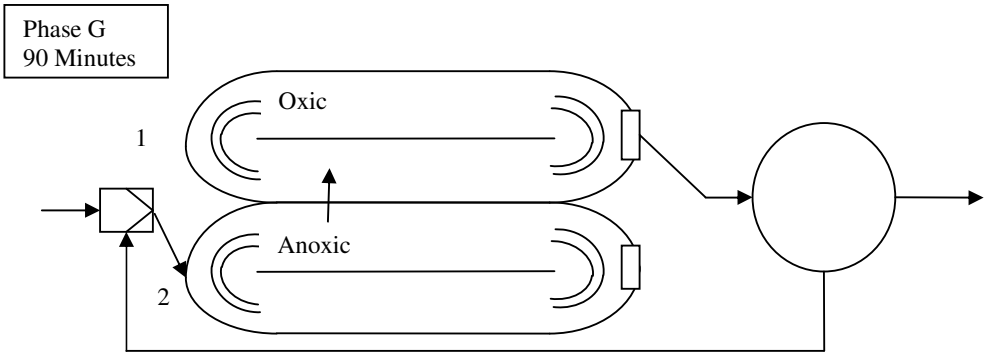

Figure 4: Ditch 1 - nitrification (oxic), Ditch 2- denitrification (anoxic)

Phase J

The cycle will end with the end of operation of phase J. Phase J has duration of 30 minutes and is the mirror image of phase E. Ditch 1 will remain in nitrification mode of operation during this phase and influent flow will be directed to ditch 1. Ditch 2 is kept in oxic condition by turning the rotors in ditch 2 during this phase, so that the ammonium concentration in ditch 2 will decrease due to oxidation of ammonium (nitrification). As a result of nitrification, the nitrate concentration will increase.

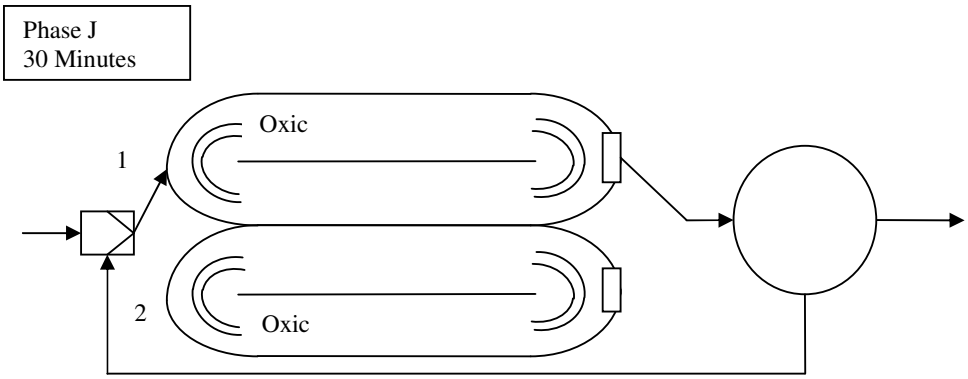

Figure 5: Ditch 1- nitrification (oxic), Ditch 2 – nitrification (oxic)

The clarified effluent from the secondary clarifiers will undergo further treatment steps such as filtration and disinfection. The sludge will either be wasted or will be returned to anaerobic selector (basin). A selector is a reactor or basin with an environmental condition such as lack of DO, food etc that favours the growth of a particular group of species of microorganisms over others (Henze *et al*, 1995)

The cycle will be completed with phase J. The weir in ditch 1 will be raised and weir in ditch 2 will be lowered and again another 4 hour cycle of operation will begin (Source: Princeton Indiana, 2004).

3.3.2.3 Key Reactions in Biodenipho process

The biological phosphorus removal (BPR) and biological nitrogen removal are the main removal processes that are carried out during the biodenipho process. Biological phosphorus removal is explained in detail in the preceding section 4.0.

Biological nitrogen removal is a process in which nitrogen is removed in two-stage process that is nitrification followed by denitrification. Nitrogen removal process like this in which nitrogen is removed from water as nitrogen gas is called dissimilative nitrogen reduction (Gillberg *et al*, 2003). During biological treatment nitrogen is also removed via sludge as some of the nitrogen is taken up by biological sludge. Around five grams of nitrogen is removed when 100 gram of sludge production is removed as biological sludge during biological treatment. This type of nitrogen removal is called assimilative nitrogen removal. In wastewater, nitrogen is mostly in the form of ammonium (NH_4^+). Nitrification is the conversion of ammonium into nitrate by the help of autotrophic bacteria by using oxygen. These bacteria oxidise ammonium to nitrate in presence of oxygen in two stages as shown in equation (1) and (2) (Gillberg *et al*, 2003).

$$NH_4^+ + 1.5\ O_2 \rightarrow NO_2^- + 2H^+ + H_2O \qquad (1)$$

$$NO_2^- + 0.5\ O_2 \rightarrow NO_3^- \quad (2)$$

Adding equation (1) and (2) gives equation (3)

$$NH_4^+ + 2\ O_2 \rightarrow NO_3^- + 2H^+ + H_2O \quad (3)$$

It can be seen from these equations that nitrification produces acid which will drop the pH if the alkalinity of wastewater is low (Gillberg *et al*., 2003).

In equation (1), ammonium is oxidised to nitrite by a group of bacteria and nitrite is oxidised to nitrate by another group of bacteria as shown in equation (2) (Henze *et al*, 1995). This group of bacteria which transforms ammonium to nitrite is also often called ammonium oxidizers and another group of bacteria which oxidises nitrite to nitrate are often called nitrite oxidisers (Philips *et al*., 2002). The nitrifying bacteria are autotrophic bacteria (Gillberg *et al*, 2003).

Nitrification is influenced by factors such as substrate concentration, temperature, oxygen, pH and toxic substances etc. Nitrification works best at a pH range of 8 to 9 and nitrification process completely stops working if pH falls below 5.5 (Henze *et al*, 1995). Nitrifying bacteria are also sensitive to temperature. Within the temperature range of 8 to 30°C the growth rate of nitrifying bacteria increases. For nitrification, optimal temperature range is from 28 to 32°C. Nitrification ceases for temperature lower than 5 °C and also for temperature higher than 45°C (Gerardi, 2003). Dissolved oxygen (DO) also plays an important criteria as nitrifying bacteria are strict aerobes and capable to nitrify only in the presence of oxygen. Within the DO range of 0.5 to 1.9 mg/l, nitrification is accelerated. For DO concentration of 2 to 2.9 mg/l, significant nitrification occurs. Maximum nitrification occurs near a DO concentration of 3.0 mg/l. However if a higher DO concentration is maintained in the aeration tank and cBOD is removed more efficiently due to the high DO, additional nitrification can be achieved (Gerardi, 2003). Nitrification can be inhibited by substances such as metal ions concentration, organic materials like sulphur components, aniline components, phenol and cyanide etc (Gerardi, 2003).

Denitrification is a process in which microorganisms reduce nitrite and nitrate to nitrogen gas in absence of oxygen while oxidizing organic matter. The reaction formula for denitrification is shown in equation (4) (Gillberg *et al*., 2003).

$$2NO_3^- + H^+ + \text{organic matter} \rightarrow N_2 + HCO_3^- \quad (4)$$

Denitrification will increase the alkalinity as seen in equation (4). Half of the alkalinity lost during nitrification process will be recovered. Most denitrifying bacteria are facultative in nature as they prefer to use oxygen as electron accepter rather than nitrate when oxygen is available. So, if oxygen is present it will deteriorate the denitrification process. Anoxic condition is the condition required for denitrification, so oxygen must be excluded from the denitrification process. Anoxic condition means absence of DO but it must contain oxygen bound up as nitrate. These denitrifying microorganisms are heterotrophic and they need organic carbon as substrate (Gillberg *et al*., 2003).

For the removal of one gram of nitrogen it will require a carbon source equivalent to 3-6 grams of COD. Internal carbon sources mean the organic content of wastewater which will be used for nitrate reduction. Dissolved organic fraction gives the highest rate of denitrification.

But this organic fraction is not usually sufficient for reduction of 50% in total nitrogen (Gillberg *et al*., 2003). So if the retention time is long, the utilization of the less accessible internal carbon source such as particulate organic matter in the wastewater can be done. In order to achieve higher reduction rate, the organic fraction of precipitated sludge obtained by biological or chemical decomposition through sludge hydrolysis can be either used as carbon source or external carbon source such as methanol, ethanol, acetic acid or starch may also be added to the process. The optimum pH for denitrification is within 7 and 9. Denitrification is also a temperature dependent process but less, compared to nitrification process (Gillberg *et al*., 2003).

In the Biodenipho process, as the return sludge is passed directly into the anaerobic tank, it is important to ensure that there is low nitrate or zero nitrate in the return sludge by sufficient denitrification process. The efficiency of biological phosphorous removal relies on the substrate concentration in the wastewater. (ISIWIKI, 2006).

3.3.2.4 ATS (Aeration tank settling) operation in the Biodenipho WWTP

The aeration tank settling (ATS) is an operation where the aeration tanks of alternating plants are introduced with settling periods and are allowed to store suspended solids (SS) during rain storms (Bechmann *et al*., 2002). In dry weather condition, as the aeration tanks remain fully mixed, the SS concentration in the effluent is same as that in the aeration tank. But during ATS operation, the effluent will be taken out from the aeration tank where sludge settles and therefore the SS concentration in the effluent will be lower than the average SS concentration in the aeration tanks. ATS increases the hydraulic capacity of the WWTP. ATS operation is activated in the WWTP during rain storms conditions and the aeration scheme is changed. During ATS operation the influent will be directed to the aerobic tank (nitrification tank) and effluent will be taken out from the anoxic tank (denitrification tank) (Bechmann *et al*., 2002). When the mixers are switched off in the anoxic tank, settling occurs. Effluent will be taken out from this tank where sludge settles to the clarifier (secondary settler). By this operation more SS can be kept in the aeration tank and hence the SS load to the clarifier is decreased. High SS concentration in the effluent will limit the hydraulic capacity of the clarifiers and give rise to high SS concentration in the effluent discharged to the receiving waters. Hence it is important to keep the SS in the aeration tanks as much as possible during rain storms. The SS concentration in effluent from aeration tanks can thus be reduced further by introducing intermediate phase with settling and anoxic conditions in both the tanks (Bechmann *et al*., 2002). The effluent SS concentration from the aeration tanks can be optimized by efficient control of flow path and settling thereby not limiting the pollution load capacity in terms of COD or BOD flux unnecessarily in the treatment plant. In WWTPs, the ATS operation is activated based on the flow measurements and prediction of the influent flow to the WWTP. The treatment plant can be prepared before the storm flow reach the plant. So sludge recirculation from the secondary settlers to aeration tanks will be increased before the influent flow is increased. In this way, SS will be decreased in the secondary settler and will be increased in the aeration tanks. But as the storm water enters the plant, the sludge recirculation will be reduced to lower level (Bechmann *et al*., 2002).

3.3.2.5 Secondary Clarifiers

Settling tanks following the biological treatment are secondary settlers (Hammer, 1986). To separate MLSS by gravity from the mixed liquor is the main purpose of these clarifiers. The circular secondary settler may be of peripheral-feed design or centre-feed design (Boyle *et al*., 2004). Here a peripheral-feed/peripheral overflow (PF/PO) clarifier is only defined since the same type of clarifier exists in Lundåkraverket WWTP.

The flow is introduced into the tank through the channel which is surrounding the periphery of the tank (Princeton Indiana, 2004). Orifice spacing in the feed channel floor provides controlled head loss with uniform flow distribution and also helps in preventing deposition of solids on the channel floor. The controlled flow enters the tank uniformly though the orifices at low velocities (Princeton Indiana, 2004). The velocity is controlled by a baffle. As the flow moves outwards, up and reaches the peripheral effluent channel in a circular motion, full volume of the tank is utilized. As influent and effluent channel are on the periphery of the tank, it permits effective skimming of the entire tank surface and influent raceway. The scum baffle prevents the surface scum collected to enter the effluent channel. The blade mounted on as extension of skimmer arm acts as skimmer in the influent channel. The collected scum is also driven by the skimmer arm to the weir gate (scum box) for removal. The scum is removed from the channel by lowering this gate (Princeton Indiana, 2004). Settled solids are collected in truss supported unitube headers (Boyle *et al*., 2004). It is then conveyed to a closed manifold which rotates around the centre support pier. Through the slotted opening beneath the manifold, the return activated sludge drops to RAS piping. For individual tank control and flow monitoring, there is a separate pump valve and parshall flume provided in the sludge pipe from which the sludge moves to a screw pump lift station (Boyle *et al*., 2004).

A well dimensioned settling tank is essential for a biological phosphorus removal process where phosphate is stored in the sludge. Increase of phosphorus in effluent might be caused by following processes:

Washout of suspended solids with effluent

In the bio-P process where there is high P content in activated sludge, the P content in the suspended solids will also be relatively high and leads to high P concentration in the effluent. So it is important to keep the suspended solids concentration as low as possible in order to minimize the total effluent P content.

Anaerobic condition in secondary settler

It is important to prevent sludge being transferred to anaerobic conditions from anoxic condition during sludge settling in the secondary settler, because during anaerobic condition phosphate will be released and leads to P content in the effluent. The secondary unwanted P release is influenced by the following factors:

Sludge retention time

The sludge retention time (SRT) or sludge age may be defined as the average time the microorganisms remain within the system. Sludge age is the ratio of the mass of organisms in the reactor to the mass of organisms removed from the system each day (Mulkerrins *et al*., 2004). Limitation of SRT in the secondary settlers can prevent the secondary P release in the secondary settlers. Too high sludge retention time will prevail when there are disturbances in process operation such as too low return sludge flow and also in case of very large settlers. So sludge is centrally removed in large settlers to reduce the sludge retention time (Janssen et al. 2002).

Oxygen and nitrate content

The presence of oxygen and nitrate will help slow down the transition of sludge to get into anaerobic conditions. So, aeration of sludge before it enters the settler will retard the P release process. Also by maintaining low sludge level will help in preventing unwanted P release (Janssen *et al*. 2002).

Turbulence between sludge level and effluent
The design of secondary settler must be done in order to limit the turbulence and exchange between sludge and water phase. Turbulence in sludge level will cause the movement of phosphate ions to upper water level causing higher P content in effluent (Janssen *et al*. 2002).

By performing endogenous P release test, one can collect the information about the P release magnitude in the secondary settler (Janssen *et al*. 2002).

3.3.2.6 Advantages of Biodenipho Process

In this process depending on seasonal variations, weekly, monthly or daily variation operation of the phases can be adjusted. For example, for summer low flow periods, the anoxic phase can be extended to increase denitrification and hence also helps to reduce energy and cost for aeration. Similarly, during winter period aerobic phase can be extended in order to maintain complete nitrification well (ISIWIKI, 2006).

Recycling of nitrate from aerobic to anoxic tank is not needed because of alternating processes and hence will help save some energy (ISIWIKI, 2006).

The key reaction rates can be estimated by monitoring concentration gradients with respect to time in aeration basins due to semi-batch manner of operation (Isaacs *et al*., 1994).

The activated sludge is passed to secondary settler from aerobic zone which will in turns maintain the low orthophosphate concentration in the effluent. Using various computer programmes a solution for the process control on online ammonium and nitrate is also possible (Janssen *et al*., 2002).

For the denitrification process a carbon source is needed which is provided by the influent wastewater in Biodenipho process, as the influent wastewater is directed to anoxic ditch most of the time (Princeton Indiana, 2004).

In the wastewater treatment plants operating in Biodenipho process, phase isolation of ditches are done based on the influent wastewater composition and hence the nitrification and denitrification volume can be modified when needed. Also the alternating modes of operation provide flexibility for operators in customizing the process to accommodate the specific characteristics of wastewater being treated by means of phase control (Princeton Indiana, 2004).

By the utilization of anaerobic selector, the growths of filamentous organisms are prohibited. Hence it helps to ensure optimum settling conditions in secondary settlers (Princeton Indiana, 2004).

Kruger implements the dissolved oxygen (DO) controlled aeration operation in the Biodenipho process. This ensures the actual volume of oxygen required to maintain oxic condition. This DO control helps to minimize the energy consumption by turning off the rotors when enough oxygen is present in the wastewater, and using only submerged mixers for keeping activated sludge in suspension (Princeton Indiana, 2004).

3.3.2.7 Disadvantages of Biodenipho Process

High investment costs as mixers and aerators must be equipped identically in both the tanks (ISIWIKI, 2006).

In order to control the process skilled personnel are needed because usually there are lots of online instrumentations installed (ISIWIKI, 2006).

3.4 Final/Tertiary Treatment

Tertiary treatment can be added to Secondary treatment for further removal of gross pollution such as BOD, SS and phosphorus to lower levels (Brett *et al*., 1997). There are different forms of tertiary treatments in wastewater, but here, the discussion is limited only to flocculation and lamella sedimentation keeping in mind about the actual situation at Lundåkraverket WWTP.

3.4.1 Flocculation

Coagulation-flocculation processes helps to remove suspended solids and colloids. Adding of mineral salts or organic compounds in the water causes agglomeration of these particles and they can be removed by allowing them to settle or by filtration. Mainly aluminium and iron salts are used as coagulant for dosing (Carballa *et al.*, 2001). Aluminium and iron salts posses the ability to aggregate particles to a size which can be removed. Almost all particles in natural water are charged, of which most of them are negatively charged. So these negatively charged particles remain in repulsion and dispersed unless they find something to get adsorbed to. Aluminium and iron ions are positively charged which forms polymer ions in which hydroxide will bind to small particles and flocs are formed (Gillberg *et al*., 2003). These flocs are allowed to settle and are removed. In order to remove metal ions coagulants should be added in high turbulence so proper mixing is important. So by this method turbidity of water is removed. Generally flocs are formed to a limited size depending on the coagulant dose, type of coagulant used, pH, temperature etc (Gillberg *et al*., 2003). For removing positively charged toxin ion flocculation should be done in high pH range. Because at high pH, greater number of negatively charged ions are gained by hydroxides and they can bind with positively charged ions and it will be accumulated in the sludge (Gillberg *et al*., 2003). Ferric salts work better than aluminium salts for slightly lower pH range and reduces the pH of treated water more compare to aluminium salts (Gillberg *et al*., 2003). The chlorides salts can work in a broader pH range than sulphate salts (Gillberg *et al*., 2003).

3.4.2 Lamella sedimentation

In lamella sedimentation tank, the well flocculated water is passed through the lamella sedimentation tank where separations of solids and liquids are done. Solids as the chemical sludge are removed from the bottom when needed and taken for sludge treatment while the clarified effluent will be discharged (Stantec Inc).

4.0 Biological phosphorus removal

4.1 Introduction

Phosphorus (P) is considered to be a key nutrient which is responsible for stimulating the growth of algae and other photosynthetic microorganisms such as toxic cyanobacteria (blue-green algae) (Oehmen *et al*., 2007). Although nitrogen and phosphorus are the limiting factors for surface water eutrophication, the contribution of 1kg phosphorus is as much as that of 10 kg nitrogen for eutrophication (Baetens, 2000). It has also been noticed that when the phosphorus concentration is reduced to 8-9 µg P/l while the concentration of nitrogen is 4-5 mg/l in large reservoirs, no eutrophication occurred (Baetens, 2000). That is why it is essential to remove phosphorus from wastewater before being discharged to aquatic water systems. Stringent effluent criteria have been imposed by the legislation during last decades in many parts of world and this has led to the construction of many treatment plants with complicated flow scheme and process layout. In Sweden, most treatment system with phosphorus removal are either chemical plants or the biological system combined with chemical treatments (Baetens, 2000). Enhanced biological phosphorus removal (EBPR) is a process that promotes the removal of phosphorus without any addition of chemicals (Oehmen *et al*., 2007). EBPR is often referred to as bio P process. A suitable process configuration, bio P bacteria and easily degradable carbon source such as volatile fatty acids (VFA) is required for the bio P process (Tykesson, 2002).

4.2 Principle

It is believed that there are some bacteria that can store considerable amounts of orthophosphate in their cells which remain in the form of insoluble polyphosphate. These bacteria, are often referred to as phosphorus accumulating organisms (PAOs) and it is these PAOs which form the backbone of the bio P process (Janssen *et al*., 2002). EBPR can be achieved by making a recirculation of activated sludge though anaerobic and aerobic condition in an activated sludge process (Oehmen *et al*., 2007). These PAOs can absorb and store an amount of phosphorus in excess than the quantity required for its metabolism and cell growth. Thus, the wastewater is treated by creating suitable environment for the presence and growth of PAOs. In this bio-P process, the mixture of influent wastewater and the activated sludge containing PAOs are at first taken to a period of incubation in an anoxic zone and then it is subjected to a period of incubation in an anaerobic zone where volatile fatty acids (VFAs) are added. The mixture is then passed to aerobic zone. The wastewater from the aerobic zone is then allowed to settle in the clarifier and the clarified wastewater is discharged as effluent (Nungesser, 1995). On the other hand, the settled activated sludge which is rich in phosphorus content is removed from the clarifier and a small part this sludge is wasted. The remaining part of the activated sludge is returned to the system and mixed with influent water in anoxic zone (Nungesser, 1995).

4.3 Mechanism

The influent wastewater must be mixed with activated sludge in a strictly anaerobic zone (absence of nitrate/oxygen), for the bio P process to occur. The influent wastewater contains acetate or VFAs or is formed by hydrolysis and acidification in anaerobic phase. The advantage of PAOs to grow in relation to the non-PAO organisms is their stored polyphosphate which they use as energy back up (Janssen *et al*., 2002). During anaerobic

condition, the splitting of stored polyphosphate takes place which results in phosphate release in the water phase as orthophosphate. The energy obtained from the splitting of polyphosphate is utilized by the PAOs, in taking up substrate under anaerobic condition in the form of acetate or VFAs and store it as carbon containing storage products such as PHB (poly-hydroxy-butyrate). Under subsequent aerobic or anoxic condition, the carbon reserves stored as PHB are oxidised with oxygen or nitrate. The released energy during this process is used by PAOs in taking the orthophosphate from the water phase and stores it as polyphosphate in their cell for growth. There is lower or no substrate (acetate or VFAs) available for organisms other than PAOs during anaerobic and anoxic phase. So by maintaining a constant activated sludge quantity and by removing the excess sludge, phosphate is removed from the treatment system (Janssen *et al*., 2002).

In brief, in the bio P process during anaerobic phase, VFAs are decreasing and orthophosphate (Ortho-P) are increasing (which is also called P-release) in the water phase as shown in Figure 6, but in biomass as shown in Figure 7, carbon storage such as PHB is increasing and polyphosphate (Poly-P) is decreasing which is also called P-release. In aerobic phase/anoxic phase, orthophosphate decreased from water phase as shown in Figure 6, whereas in Figure 7, it is seen that polyphosphate is increased which is also called P-uptake and PHB is decreased in the biomass. Hence the phosphate is removed from water phase in wastewater treatment (Janssen *et al*., 2002).

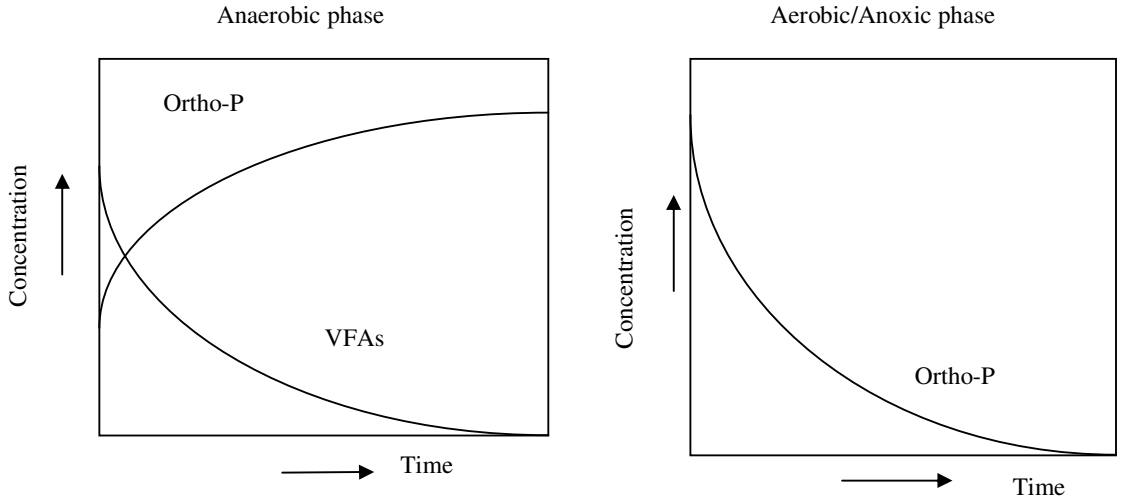

Figure 6: Anaerobic and Aerobic/Anoxic phase in bio P process in the water phase

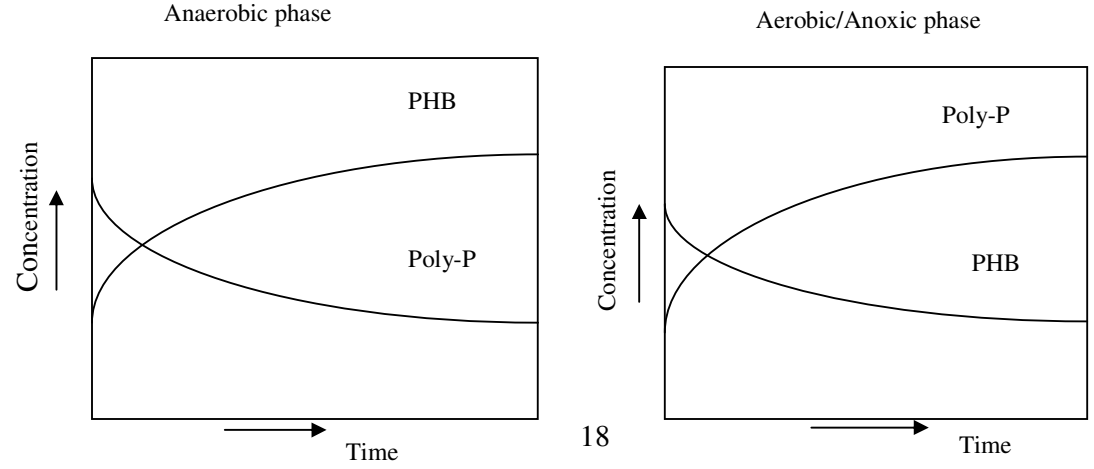

Figure 7: Anaerobic and Aerobic/Anoxic phase in bio P process in the biomass

4.4 Microbiological aspects of bio P process

It is believed that there are mainly two groups of bacteria that are able to take larger amounts of phosphate. They are mainly Poly-P organisms such as *Acinetobacter* and *Microthrix parvicella* which can store phosphate in the form of poly-P for maintenance purposes and Phosphate accumulating organisms (PAOs), which are the 'real' phosphate bacteria which store polyphosphate in the anoxic and/or aerobic zone (Janssen *et al*., 2002). The advantage of the PAOs over the other group is that, though both the groups can survive in the anaerobic phase due to their polyphosphate reserves, the PAOs are dominant due to their ability of taking up substrate with the energy from polyphosphate (Janssen *et al*, 2002).

In the recent developments new molecular methods such as Fluorescence in situ hybridisation (FISH) have been used to study the PAOs. The labelled PAOs in an activated sludge can be observed using a confocal laser scanning microscope (CLSC) or an epifluorescense microscope in combination with some other method such as micro auto-radiography (MAR) where the uptake of the radiolabelled organic substrate or phosphate can be shown. By using this FISH studies it was found that the *Rhodocyclus*-related bacteria within the *β-Proteobacteria* plays an important role in the EBPR process (Hesselmann *et al*., 1998 and Crocetti *et al*., 2000 cited in Tykesson, 2002). Research carried out by Lee *et al* (2002 cited in Tykesson 2002) also found that this group of bacteria was different in two pilot plants with different configurations which proved that it is not a single group of bacteria that can be held responsible for the total EBPR in each system.

4.5 Biochemical aspects of bio P process

The metabolism of the bio-P process is dependent mainly on three storage compounds: polyphosphate, glycogen and poly-hydroxy-butyrate (PHB) (Janssen *et al*., 2002). The conversion of acetates to PHB in the anaerobic phase needs a reduction capacity in the form of $NADH_2$ as PHB is a more reduced polymer than acetate. $NADH_2$ is formed by the conversion of glycogen into PHB in the PAO cell and the required energy is obtained by the hydrolysis of polyphosphate with a decrease in orthophosphate concentration in the activated sludge (Janssen *et al*., 2002). Therefore, it is seen that both glycogen and polyphosphate is responsible for the maximum P release in addition to acetate (Janssen *et al*., 2002). Another study reveals that glycogen can be a limiting substance in anaerobic substrate uptake only during shock loading conditions (Mino *et al*., 1998). This is because when excess acetate is fed, the exhaustion of glycogen stops the anaerobic uptake of acetate by PAO enriched sludge. It has also been further studied that poly-P can also be a limiting substance at high pH, because a high pH requires a higher energy for acetate transport through the membrane (Mino *et al*., 1998).

In the aerobic or anoxic process, PHB is oxidised to CO_2 and $NADH_2$ is released, which is converted to ATP (Adenosine tri-phosphate). The PAOs use the energy from the ATP to grow, to store orthophosphate as polyphosphate in the cell and to form glycogen (Janssen *et al*., 2002). During the phosphate uptake under aerobic conditions, oxygen is used for the formation of ATP, whereas during anoxic conditions nitrate is used. However under anoxic conditions, formation of ATP is approximately 40% less per amount of $NADH_2$, which ultimately results in lower biomass production in anoxic conditions. This is directly related to

the production and growth of PAOs and does not have a big influence on the total sludge production (Janssen *et al*., 2002).

There will be a negative effect on P-uptake if there is presence of phosphate and carbon source in both aerobic and anoxic conditions (Mino *et al*., 1998). Because carbon sources available under these conditions will be used for conversion into carbon storage products such as PHB and no P-uptake will take place unless carbon sources are exhausted. So the situation where carbon source still available in aerobic and anoxic condition should be prevented (Mino *et al*., 1998).

Glycogen degradation during the period of 'over-aeration' will lead to decrease in capacity of PAOs to store substrate during anaerobic condition and hence will have negative effects on aerobic P-uptake (Janssen *et al*., 2002). Over-aeration occurs when it is aerated for too long such as after long periods of rain or during period of under-loading (such as during low loading periods in weekends) and under these condition PAOs will oxidise glycogen after a rapid oxidation of PHB (Janssen *et al*., 2002).

4.5 Glycogen accumulating organisms (GAOs)

Unwanted growth of glycogen accumulating organisms (GAOs) can be a potential reason for EBPR failure. GAOs can compete with PAOs for carbon source. Like PAOs, under anaerobic condition these GAOs are also capable of taking the VFA and convert it into PHA (poly-β-hydroxyalkanoates) (Oehmen *et al*., 2006). When acetate is the only carbon source in anaerobic condition then the polymer PHA is also called PHB (Mino *et al*., 1998). But unlike PAOs, these GAOs do not release P in anaerobic condition and neither do they uptake P in aerobic/anoxic conditions and hence it does not contribute in phosphorus removal (Oehmen *et al*., 2006). They hydrolyse glycogen for the source of energy required for anaerobic VFA uptake. As GAOs do not contribute to phosphorus removal and only consumes the valuable VFAs in wastewater, it is considered to be unwanted organisms for EBPR systems. For efficient EBPR process the selection of PAOs over GAOs should be done (Oehmen *et al*., 2006). According to recent studies the composition of VFA will provide the selection pressure. Oehmen *et al*. (2006) suggested that propionate provide PAOs competitive advantages over GAOs and are considered to be more efficient carbon source than acetate for EBPR process. His studies further revealed that in full scale EBPR plant the wastewater consists of both acetate and propionate and is potentially advantageous for PAOs as they can use both the substrate whereas GAOs prefer a more specific carbon source. He also suggested that reliability and efficiency of EBPR process can be improved by increasing the propionate fraction in wastewater by operating a prefermenter or by addition of propionate as supplementary carbon source.

4.6 Factors affecting biological phosphorus removal

4.6.1 Temperature

Temperature is an important factor in the bio P process. The change in temperature, either an increase or a decrease has an influence in various levels of activated sludge process (Janssen *et al*., 2002). The effects of temperature are seen not only in the metabolic activities of the microbial population but also on factors such as gas transfer rates and settling characteristics of the biological solids (Mulkerrins *et al*., 2004).

With approximately every 10°C increase in temperature, it has been observed that the growth rates double. Studies carried out on temperature changes have conflicting effects in the overall EBPR. In one study it was observed that the biological phosphorus removal efficiency improved in higher temperatures such as 20-37°C and on the other better P removal efficiencies have been found in lower temperatures such as 5-15°C (Mulkerrins *et al*., 2004). In another study it was observed that plant efficiencies are affected in the northern hemisphere during winters since temperatures are below 0°C. Also sudden change in temperature and hydraulic loadings can cause serious problems in the plant with long term effects resulting in the deterioration of the EBPR performance for weeks (Mulkerrins *et al*., 2004).

Oxygen consumption rates are also affected by temperature changes. In a study conducted, it was observed that at 5°C and 10°C, P-uptake was incomplete in the aerobic phase and at 20-30°C complete P-uptake was observed. P-release reduces with a decrease in temperature with sludge activity being highly affected by quick temperature changes. Overall, it is seen that P-release and P-uptake increase with increasing temperatures and vice versa (Mulkerrins *et al*., 2004).

It was reported that temperature had a strong effect on the accumulation of storage polymers, with less PHB formation in high temperatures. It was also found that with increase in temperatures from 15 to 35°C, the rates of substrate uptake, ammonium consumption, oxygen uptake, CO_2 production and PHA formation all increased (Mulkerrins *et al*., 2004). An important reason for the negative effects of low temperature may be due to the fact that bacteria such as *Microthrix parvicella* causes sludge bulking with an optimum growth of this organisms in the activated sludge at temperatures less than 12-15°C (Mulkerrins *et al*., 2004).

4.6.2 pH

pH is an important factor to be considered in the EBPR process and therefore it needs to be carefully monitored time to time. The different processes such as nitrification, denitrification, P-release and P-uptake all have specific pH ranges (Mulkerrins *et al*., 2004). During the anaerobic phase, transport of acetate into the cell is influenced by operating pH. Low pH requires more acetate per amount of phosphate released (Janssen *et al*., 2002).

Another influence of pH is the physical-chemical bonding of phosphate in the activated sludge. The solubility product of metal-phosphate compounds exceed at pH values higher than 7.5 which results in precipitation of the metal phosphates. This supports the total phosphorus removal in the bio-P process which depends on the concentration of the cations in wastewater and the prevailing pH conditions. The presence of higher cations and orthophosphate concentrations in the anaerobic phase contribute to the stimulation of the spontaneous precipitation in the bio-P process. It is seen that pH values higher than 7.5 in the activated sludge process supports P reduction in two ways: through increased polyphosphate uptake and through chemical precipitation (Janssen *et al*., 2002).

4.6.3 Wastewater composition

Wastewater composition is an important factor in determining the efficiency and operation of the bio P process. Also disturbance in wastewater due to dilution, as in times of heavy rainfall may cause prolonged problems in the EBPR system. Accumulation of GAOs must also be avoided which may be caused due to changes in the influent organic composition from VFAs to sugars such as glucose (Mulkerrins *et al*., 2004).

The disturbances in the inlet wastewater concentration such as dilution of rain water due to storm events and back to normal wastewater loading, has an impact causing increased phosphate concentration in aerobic/anoxic reactors and the effluent from the plant (Kruhne *et al*., 2003). The different recovery times for aerobic/anoxic P-uptake and anaerobic P-release is thought to be the reason for this phenomenon to occur. The PHA stores in the bacteria are depleted due to less available soluble organic components during the rain event of low strength inlet wastewater. When the influent wastewater returns to normal strength, the recovery of P-release takes place instantaneously whereas P-uptake will not be fully recovered unless the PHA stores are replenished (Kruhne *et al*, 2003). Such temporary imbalance between P-release and P-uptake during the recovery from such perturbation shows high phosphate concentration in the effluent (Temmink *et al*., 1996). An experiment was performed by Temmink *et al*. (1996) and he concluded from this experiment that the P-release and PHB stores gets recovered instantaneously whereas the response of P-uptake seems to depend on slowly increasing level of PHB. For EBPR recovery, a careful adjustment of aeration time to the organic loading is required mainly at low organic loading conditions (Temmink *et al*., 1996). High concentration of internally stored carbon is possible to maintain by addition of external carbon source or by reduction of aeration time during such disturbed conditions (Kruhne *et al*., 2003). During such disturbed conditions the wastewater has lower concentration of NH_4-N, PO_4-P as well as influent organic concentration. Because of this, the necessary aeration time for nitrification and biological phosphorus uptake is also reduced. So, by reducing the aeration time the unnecessary consumption of PHA can be lowered (Kruhne *et al*., 2003). Kruhne *et al*. (2003) also concluded from his experiment that, addition of external carbon source such as sodium acetate in the influent anaerobic reactor or into the anoxic phase of the process could improve the BPR process. He further observed that BPR process got stabilized at low internal stored PHA levels in cases of anoxic addition whereas BPR process failed in case of the anaerobic addition even though high quantity of PHA were available. He concluded that this observation could possibly be due to low concentration of PAO in the system and not due to PHA limitation. From his experiments he also concluded that by the application of an addition of low external carbon source along with a reduced aeration time, it was possible to stabilize the BPR process.

It has been found that COD loading also affects the EBPR process. In a study using a biofilm system, it was found that anaerobic P release was quite efficient with increase in influent acetate concentration up to 400 mg/l. But a further increase up to 600 mg/l leads to cessation of the anaerobic P release and deterioration of the P-removal capability (Mulkerrins *et al*., 2004). High influent concentrations also have negative effects in the bio-P process. Sludges with low COD-SS loading rates display high P-uptake potential and that with high COD-SS loading rates display low P-uptake potential. Actually, during higher COD-SS loading rates, influent organic matter is converted to storage product 3-hydroxyvalerate which is utilised by the GAO bacteria which further worsens the EBPR. Decrease in organic loading can also cause a simultaneous increase in the effluent nitrate concentrations thereby causing deterioration of the EBPR (Mulkerrins *et al*., 2004).

Pre-settling will remove a part of the COD from wastewater. This will affect the normal incorporation of phosphate in the activated sludge thereby lowering the organic bound phosphate (Janssen *et al*., 2002).

Higher BOD:P and BOD:N ratio makes it simpler to remove phosphate and nitrate biologically. Under conditions where the amount of nitrate recycled to the anaerobic zone is negligible, the BOD:P ratio should be at least 15-20 to guarantee bio-P removal. Also the

BOD:N ratio must be at least 4-5 to ensure good functioning of bio-P and N removal (Janssen *et al*., 2002).

Low concentrations of potassium and magnesium in the influent may also cause problems in the bio P process. They act as counter-ions for the negatively charged phosphate ions. Phosphate uptake is therefore coupled with potassium and magnesium uptake (Janssen *et al*., 2002).

4.6.4 Sludge loading and sludge age

An increase in sludge loading and corresponding decrease of sludge age will result in higher sludge production. So this will lead to increase in the removal of a 'normal' organically bound phosphate through excess sludge and hence less phosphate is needed to be removed via the process of storage of polyphosphate (Janssen *et al*., 2002). Whereas vice-versa condition occurs when there is a (very) high sludge age, that is decrease in the discharge of a 'normal' organically bound phosphate through excess sludge and hence requires much of the phosphate to be removed through storage of polyphosphate (Janssen *et al*., 2002). Decrease in sludge age due to high sludge loading results in decrease in nitrification and reduces the inhibition of bio-P caused by nitrate and hence storage capacity of polyphosphate is increased. The minimal aerobic/anoxic sludge age necessary for P-uptake is less than 3 days (Janssen *et al*., 2002). A (very) high sludge age increases a chance of excessive aeration during low loaded condition. So this over-aeration may cause inhibition of bio-P and growth of PAOs because of storage compound including glycogen of PAOs being oxidised. A very high sludge age also causes mineralization of PAOs in the activated sludge process which releases polyphosphate in a soluble form (Janssen *et al*., 2002). According to case studies done by Domokos *et al*. (2005) with the aid of computer simulation of a treatment plant facing occasional drastic phosphorus peaks, he concluded that the problem was caused by the undesirably long retention time in the settler which led to the phosphorus dissolution from the sludge in the settler into the effluent causing high phosphorus concentration.

It is important to monitor SVI and MLSS concentration in order to know the settling properties of the sludge (Mulkerrins *et al*., 2004). As lower the sludge index, the better is the sedimentation performance. Normally sludge index is in the range of 60-150 ml/g. A high sludge index is not good as it may cause several problems. Sludge bulking can occur due to presence of too high concentration of filamentous bacteria and makes the water treatment difficult (Gillberg *et al*., 2003). High values of MLSS imply low values of SVI. It has been recommended that MLSS for both N and P removal should be maintained between 1500-1700 mg/l (Mulkerrins *et al*., 2004).

4.6.5 Nitrate and oxygen

Nitrate and oxygen play important roles in the bio P process. They are needed in the aerobic/anoxic zone in order to store polyphosphate in the activated sludge process. But at the same time, the presence of nitrate and oxygen in the anaerobic phase through the influent and the return streams may hinder the bio P process. The presence of oxygen and nitrate in the anaerobic process will take some COD for the aerobic/anoxic conversions and therefore there won't be enough COD left for the PAOs which will result in less phosphate removal as polyphosphate. Long term presence of nitrates in the anaerobic tank will initiate the growth of normal denitrifying bacteria which will utilise the COD that is needed for the PAOs (Janssen *et al*., 2002).

4.6.6 Anaerobic conditions/contact time (Anaerobic residence time)

As mentioned earlier, for the rapid uptake of acetate by PAOs, anaerobic condition is necessary (Janssen *et al*., 2002). The anaerobic contact time is the time that the activated sludge (influent + return sludge and recycle flows) stays under anaerobic conditions (Janssen *et al*., 2002).The required anaerobic contact time depends mainly on the amount of readily biodegradable substrate COD available, the amount of phosphate to be biologically removed and the maximum storage capacity of PAOs (Janssen *et al*., 2002). The anaerobic residence time should be such that, it is not long enough to promote the formation of sulphides and other noxious products associated with anaerobic treatment processes (Princeton Indiana, 2004). The anaerobic contact time can be determined as: when there is large amount of VFA in wastewater, the anaerobic contact time should be restricted to 0.5 hours (Janssen *et al*., 2002). If the wastewater posses a lot of raw influent that is to be acidified then the anaerobic contact time should vary between 1 to 3 hours as the primary sludge needs to acidify first. But, as current knowledge proves primary sludge in anaerobic zone doesn't contribute to a large extent in VFA production so the anaerobic contact time may be shorter (Janssen *et al*., 2002). In practice, usually it is 1 hour (Janssen *et al*., 2002).

4.7 Advantages and disadvantages of bio P process

4.7.1 Advantages of bio-P over chemical treatment

- Annual operating costs are significantly lower (Baetens, 2000).
- No chemical sludge, so less sludge production and less sludge to be treated, disposed and finally managed for (Janssen *et al*., 2002).
- It avoids strong decrease in pH which can occur during chemical dosing of acid metals and thus it prevents inhibition of nitrification caused by strong pH drops (Janssen *et al*., 2002).
- Denitrification will not be limited by the COD as in chemical treatment where large fraction of COD will be removed via primary sludge in settler by pre-precipitation (Janssen *et al*., 2002).
- Biological sludge is of better quality and can be used as fertilizer (Janssen *et al*., 2002).

4.7.2 Disadvantages of bio-P

- The efficiency of bio-P process is dependent on wastewater composition. The growth of PAOs depends on available amount and quality of organically bound carbon (COD or BOD) in wastewater (Janssen *et al*., 2002).
- The bio-P process is less stable during changing process condition such as high rainfall (Janssen *et al*., 2002).
- There is a small chance of phosphorus release during bio-P sludge treatment such as anaerobic digestion. So by thickening and dewatering, the orthophosphate released will be recycled back into the main treatment which may hamper the bio-P reduction (Janssen *et al*., 2002).
- Careful handling of sludge is important because P can be released to the environment again by the breakdown of internal P-content by the organisms (Baeten, 2000).

5.0 Bulking and foaming due to filamentous organisms (*Microthrix parvicella*)

Foam is a film of solids containing entrapped air bubbles or gases such as carbon dioxide, molecular nitrogen and nitrous oxide. When the entrapped gases and air bubbles from the foam escape, the foam collapses and scum is produced. Foam is characterized mainly by colour and texture and sludge age conditions are largely responsible for the scum. With decreasing sludge age foam becomes lighter in colour and billowy in texture, whereas with increasing sludge age foam becomes darker in colour and viscous in texture. Scum is insoluble and floats to the surface of the aeration tank and the secondary clarifier. The colour of a scum is brown and is flaky (Gerardi, 2003).

Foaming due to filamentous organisms is a common operational problem encountered in many WWTPs and is particularly been highlighted in this literature as it is one of the concerns in Lundåkraverket WWTP, Landskrona. It is caused mainly due to single or combinations of different filamentous organisms. One of the most common filamentous organisms, *Microthrix parvicella* is the focus of discussion in this section.

5.1 Harmful effects of biological foams

Foaming caused by filamentous organisms pose a number of problems in the WWTP. Some of the most common problems encountered is listed below (Madoni and Davoli, 1997):

- Difficult to control sludge concentration in the aeration tank due to trapping of a large volume of mixed liquor suspended solids (MLSS) inside the foam.
- Effluent quality is reduced due to spreading of foam over the surface of the final settlers.
- Sometimes foam may overflow the basin freeboard, thereby causing the walkovers, handrails and surrounding areas hazardous and slippery.

5.2 Physiology and growth characteristics

Microthrix parvicella is a long, thin, non branched and unsheathed filamentous bacterium with a diameter between 0.6 to 0.8 μm. One of the most distinguishing characteristic of this bacterium is its coiled appearance and characteristic gram-positive reaction (Rossetti *et al,* 2005).

5.2.1 Substrate storage

It has been studied that the organic carbon sources of *Microthrix parvicella* is solely confined to long chain fatty acids (LCFA) and it was further revealed that they can take up LCFA during anaerobic phase and they use it as future storage for their growth in the oxic and anoxic phase (Nielsen *et al, 2005*). In EBPR plants, LCFA are provided in the anaerobic plants which gives *Microthrix parvicella* an advantage over other bacteria since very few bacteria are capable of taking and storing LCFA (Andreasen and Nielsen, 1998).

5.2.2 Low DO

Low dissolved oxygen (DO) is found to be a favourable condition for the growth of *Microthrix parvicella* and good growth is obtained under microaerophilic conditions. High DO concentration greater than 6 mg/l is found to be toxic in some studies (Rossetti *et al,* 2005).

5.2.3 pH effects

pH effects on *Microthrix parvicella* growth was also studied by Slijkhuis, 1983. Tests were conducted on a pH range of 7.1 to 8.0 and it was found that during growth pH decreased from 8 to 7.6 due to acid production. It was also found that growth rate was zero at an initial pH value $\leq$ 7.1. In yet another test pH effects on the range of 6.7-8.4 seemed to be minimal on the growth rate of *Microthrix parvicella* (Rossette *et al*, 2005).

5.2.4 Temperature effects

Growth of *Microthrix parvicella* also follows a distinctive seasonal variation. Temperature was found to be one of the main factors affecting their growth rate. Temperatures below 15°C were found suitable for their accumulation and when this temperature increased their growth rate started decreasing (Hug *et al,* 2005). Similar results were obtained in another test where the growth rate seemed to increase within temperatures in the range of 7-25°C and no growth rate was observed at higher temperatures (Rossetti *et al,* 2005). It was further revealed that higher temperatures limit the availability of fats and lipids to it and also encourage other bacteria to outnumber *Microthrix parvicella* (Rossetti *et al,* 2005).

5.2.5 Effect of ammonium

Another study has found out that *Microthrix parvicella* require ammonium for their growth. In incompletely nitrifying biological nutrient removal (BNR) plants, ammonia is used as a nitrogen source for their growth and proliferation (Tsai *et al,* 2003).

5.2.6 Sludge age and sludge volume index (SVI)

As *Microthrix parvicella* is a slow growing bacteria, long sludge age that are very common in BNR plants, is proved to be beneficial for their growth (Andreasen and Nielsen, 1998).
Sludge volume index (SVI) is a measure of the sedimentation performance. The lower the SVI, the better will be the sedimentation. The normal range is between 60-150 ml/g (Gilberg *et al,* 2003). Studies have revealed that SVI values above 150 ml/g are found to be associated with filamentous bulking (Xie *et al,* 2007).

5.3 Control strategies for Microthrix parvicella

Different control strategies are available depending on the causes of the filament proliferation which are specific and some other non-specific methods treating mainly the symptoms of bulking and foaming (Rossetti *et al,* 2005). Some of the *specific methods* suggested are as follows:

5.3.1 Reducing SRT

It has been proved that high SRT promotes the growth of *Microthrix parvicella.* Therefore reducing SRT will minimize their growth (Andreasen and Nielsen, 1998). But according to latest findings, it is not recommended to decrease SRT in nutrient removal plants because doing so, it will also eliminate the nitrifying bacteria from the activated sludge (Rossetti *et al*, 2005).

5.3.2 Maintain appropriate DO concentration

Since *Microthrix parvicella* is found to be a microaerophile, maintaining a DO concentration of more than 2 mg/l in the aerobic zones is found to control their growth in some full-scale treatment plants (Rossetti *et al,* 2005).

5.3.3 Pretreatment by floatation

Lipids are also said to promote the growth of *Microthrix parvicella.* In order to reduce the high lipid contents in some wastewater it is recommended to undergo some kind of pre-treatment such as flotation. The floated material is thrown into the aerobic zone so that other bacteria will compete with *Microthrix parvicella* in the uptake of LCFA. But this method is costly because it is very difficult to separate the lipid fraction of wastewater (Rossette *et al,* 2005).

Some of the common *non-specific methods* of control are discussed below:

5.3.4 Chlorination

Chlorination is supposedly one of the most common non-specific methods for controlling bulking. Adequate amounts of chlorine must be added in order to remove *Microthrix parvicella* effectively. Lower amounts proved to be give very poor results (Xie *et al, 2007*). Also high dosing may kill the floc forming bacteria and therefore result in worse effluent qualities. So, it is also recommended to carry out microscopic examinations of the sludge in order to monitor the general effects of chlorine on the biomass (Rossette *et al,* 2005).

5.3.5 PAX-14

Dosage of polyaluminium chloride (PAX-14) is a good way to control the growth of *Microthrix parvicella* in activated sludge and at the same time it has also been observed that PAX-14 has very little or no effect on other filamentous organisms. PAX-14 is a mixture of Al_2, Al_3 and Al_{13} species and is produced from $Al(OH)_3$ at high pressure and temperature (Nielsen *et al,* 2005). Recommended dosing is between 2-3 g Al kg/MLSS/d in the recycled activated sludge stream for at least three weeks (S. Rossette *et al*, 2005). It has been found that the dosing of PAX-14 had effected *Microthrix parvicella* in a number of ways. It reduced the substrate uptake measured as an uptake of radiolabelled oleic acid and trioleic acid under anaerobic conditions (Neilsen *et al,* 2005). A reduction of exoenzyme activity was observed, particularly a reduction of the lipase activity. Finally, it favoured the formation of dense flocs due to the flocculating effect of PAX-14 which resulted in the embedment of *Microthrix parvicella* into the floc material. This ultimately resulted in the overall reduction in the availability of substrate due to increased diffusional distance (Nielsen *et al*, 2005).

It has also been recommended that it is necessary to establish a pre-alert system in order to eliminate the foaming and bulking by *Microthrix parvicella* at the beginning and thus avoid booming period (Xie *et al*, 2007).

In general it may be concluded that *Microthrix parvicella* is a highly versatile filamentous organism possessing a variety of metabolic strategies required to grow and survive under a wide range of operating conditions. It is because of this reason, it is difficult to indicate the operational cause of bulking and it is also not so easy to get a simple diagnosis for bulking in terms of kinetic and metabolic selection even after its detection in the mixed liquor (Rossette *et al*, 2005).

6.0 Description of Lundåkraverket WWTP

6.1 Introduction

Lundåkraverket WWTP is situated in the municipality of Landskrona. It is equipped with mechanical, biological and chemical treatment facilities. It was first established in 1960 and all through these years it has been upgraded with the addition of new facilities and technologies till date. In 1962 it initiated biological treatment and it is believed that it was the first of its kind in the Öresund region. In 1980, chemical treatment was also added in addition to the existing biological treatment. In 1996, it was further upgraded with biological nitrogen and phosphorus removal facilities.

Besides Landskrona municipality, Lundåkraverket WWTP is responsible for the efficient wastewater treatment of a number of other neighbouring municipalities which include Glumslöv, Saxtorp, Häljärp, Asmundtorp, Svalövs and Kävlinge municipalities. Wastewater from Ven Island is also pumped to Lundåkraverket for treatment.

Lundåkraverket serves a population of approximately 37,000 people. The incoming pump station has a capacity of 3050 m^3/hour, and the normal average flow is about 700 m^3/hour. The incoming hydraulic loading is shown in Table 2.

Table 2: Incoming hydraulic loading to Lundåkraverket

Incoming wastewater	**Hydraulic load (m^3/day)**
Domestic	9300
Industrial	1700
From other sources	2000-12000
Total dry-weather flow	13000-23000

6.2 Incoming wastewater composition

The annual average values of the incoming wastewater composition to Lundåkraverket is shown in Table 3.

Table 3: Incoming wastewater composition to Lundåkraverket (2005-2006)

Parameter	BOD_7	CODt	Tot-P	Tot-N	NH_4-N	SS
Average (mg/l)	109	333	6.0	35.4	23	221

6.3 Outlet demands

Effluent standards must be strictly followed and maintained before the treated wastewater is discharged to the receiving waters. Keeping in mind the stringent Swedish effluent standards, Lundåkraverket has put forward the following outlet demands as shown in Table 4.

Table 4: Outlet demands at Lundåkraverket

	Tot-N mg/l	Tot-P mg/l	BOD_7 mg/l
Target value	10 (yearly average)	0.4 (yearly average)	10 (quarterly average)
Limit	-----	0.5 (yearly average)	10 (yearly average)

6.4 Process description

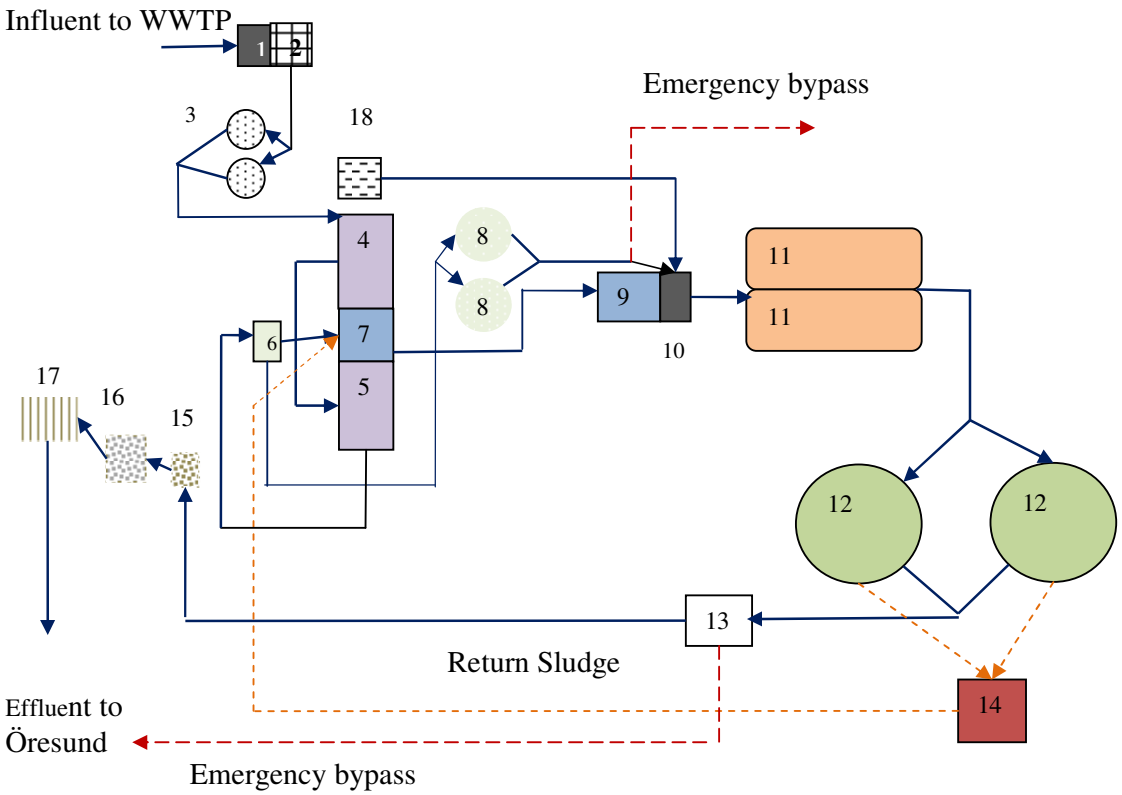

Figure 8: Block flow diagram of Lundåkraverket WWTP

1: Incoming pump station	10: Middle pump station
2: Bar screen	11: Biodenitro basin
3: Sand trap	12: Secondary settler
4: Primary settler 1	13: Bypass chamber
5: Primary settler 2	14: Return activated sludge tank
6: Distributor	15: Tank for chemical addition
7: Bio P basin 1	16: Flocculation basin
8: Trickling filter	17: Lamella sedimentation tank
9: Bio P basin 2	18: Reject water tank (from sludge treatment)

Figure 8 shows the different treatment units and flow scheme of Lundåkraverket WWTP. Wastewater is pumped into the WWTP by means of four pumps (2 big and 2 small pumps).

After that it undergoes mechanical, biological and finally chemical treatment processes. Each of these procedures has been described below step by step.

6.4.1 Mechanical treatment

Mechanical treatment at Lundåkraverket WWTP consists of bar screen and sand traps as preliminary treatment units and two pre-settlers - primary settler 1 and primary settler 2 in series connection as primary treatment units.

6.4.1.1 Bar screen

Pumped wastewater undergoes preliminary treatment with bar screens. The raw wastewater is screened by three automatic bar screens which remove rags, sticks, plastics and other debris that could damage the downstream equipments in the WWTP. Of these three bar screens, two are fine screens having 3 mm openings and a flow capacity of 1525 m^3/h and one is a coarse screen with 10 mm openings and a flow capacity of 1000 m^3/h. Total design flow of these screens is 3050 m^3/h. The two bar screens with 3 mm openings are mainly in use. The picture of a 3mm bar screen is shown in Figure 9. The debris collected in the bar screens are transported by a rotating bar cleaner and deposited in a shaft where solids are compacted and dewatered (squeezed). Hence the relatively dry solids are then discharged to dumpster for disposal to land fill. The average solid removed by screening is around 300-600 kg/day. The screened wastewater will then pass to the next preliminary treatment step, the sand trap/grit removal process.

Figure 9: A bar screen with 3mm spacing

6.4.1.2 Sand trap/ grit removal

The wastewater coming out from screening flows to two parallel sand traps. The sand trap is designed for a design flow of 3050 m^3/h, and the average flow is 700 m^3/h. There are two sand traps which are circular as shown in Figure 10 each having a diameter of 3.4 m and a volume of 30 m^3. The sand traps are of vortex type with mechanical paddle-stirring to create the circular motion. There are two paddle stirrers (propellers) one in each of the two sand traps, which is rotating with a speed of12 r/min. This circular whirling pattern creates the heavier solid particles such as sand or soil to settle down at the bottom of the aerated grit chamber (sand trap). These solid particles collected at the bottom are pumped by suction with sand pump to sand drainage. There is one sand pump for each sand trap and each of these pumps has a capacity of 0.75 m^3/min. The aeration for the grit chambers is provided by a blower machine. There are two blower machines for each sand trap and each of them has a

capacity of 2.2 m^3/min. There is only one sand drainage in the WWTP, which has 0.75 m^3/min capacity. In the sand drainage, the collected sand, grit etc. are washed and dewatered and then the solids are transported to the container for disposal to landfill. The effluent from the sand trap passes as an influent to the first primary sedimentation tank (primary settler 1).

Figure 10: Two parallel sand traps in Lundåkraverket

6.4.1.3 Primary settler 1

The primary sedimentation basin 1 is rectangular in shape and has a total area of 320 m^2 and a total volume of 500 m^3. The basin is divided into four compartments and each compartment has 80 m^2 area and 1.56 m depth. The design flow for the tank is 3050 m^3/h and average flow is 1000 m^3/h. The wastewater coming in these tanks are allowed to remain quiescent to settle out the suspended materials. The hydraulic detention time is 0.5 h. SS entering the primary settler is around 225 mg/l that is equal to around 4600 kg/d. This primary settler reduces 35% of the total SS. Therefore, SS of around 1600 kg/d will be removed as primary sludge from the bottom of the settling tank. The sludge is scraped together with plastic chain scrapers to a sludge pit located at the basin outlet and is then taken to a collecting well which is common for both pre-sedimentation basins 1 and 2. It is also possible to increase the reduction of SS, BOD and P through the addition of chemicals in the upstream mixing basin. A part of the primary sedimentation basin with scrappers is shown in Figure11.

Figure 11: Primary sedimentation basin

6.4.1.4 Primary settler 2

The effluent from the primary sedimentation basin 1 passes to primary sedimentation basin 2, where the remaining suspended materials in the wastewater are further allowed to settle. The tank is rectangular in shape and has a total volume of 1070 m^3. The tank is compartmentalized into four basins, each compartment has an area of 142.5 m^2 and depth of 1.85 m. It has a design flow of 3050 m^3/h and an average flow of 1000 m^3/h. The hydraulic detention time is 1 h. The incoming SS in this tank is around 3000 kg/d (175 mg/l), of which 45% are removed as primary sludge in this tank. Therefore, the total SS removed from this tank as primary sludge is around 1400 kg/d.

After the primary treatment (after the wastewater passes through pre-settlers 1 and 2), the effluent coming out from the pre-settler 2 will have a total reduction of 65% SS, 30% BOD_7, 2% N in SS and 1.2% P in SS.

6.4.2 Biological treatment

Biological treatment in the Lundåkraverket WWTP comprises of secondary treatment units. The part of the WWTP that deals with sludge treatment is not included in this study. The reject water from the sludge treatment is carried to the middle pump station which is attached with the bio P basin 2.

In the secondary treatment, more of the organic materials (BOD), nitrogen (N) and phosphorus (P) will be reduced. The secondary treatment comprises of the Biodenipho process and trickling filters. The Biodenipho process consists of two anaerobic basins (bio P basin 1 and bio P basin 2) in series, two biodenitro basins, and two secondary settlers in parallel connection.

6.4.2.1 Bio P basin 1

The effluent from the primary sedimentation basin 2 will pass through the distributers (No 6 in Figure 8), which is located ahead of the bio P basin 1. From there the wastewater flow will be distributed to different compartments of Bio P basin 1 as well as to the trickling filters depending upon the quantity of incoming flow to WWTP. There are four compartments in the Bio P basin 1 as shown in Figure 12. The first compartment is called the pre-denitrification tank (DN-section), the second compartment consist of two parts, the first part with small volume is the selector basin while the remaining part is the anaerobic tank (anaerobic section 1). The remaining other two compartments are strictly anaerobic tanks (anaerobic section 2 & 3). There is one mixer in each section for keeping the bio-solids in suspension. The volume of each compartment of the Bio P basin 1 is shown in Table 5.

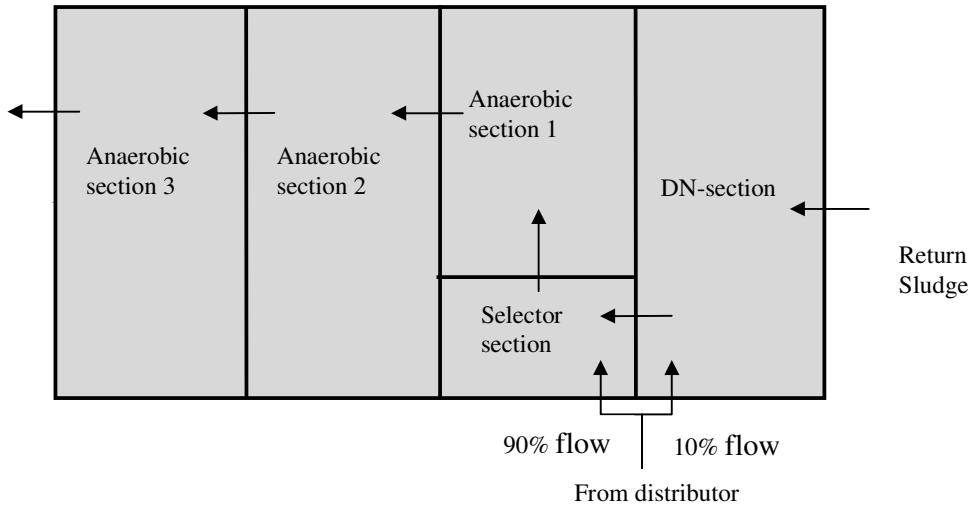

Figure 12: Schematic diagram of Bio P basin 1

Table 5: Volume of different compartments in Bio P basin 1

Bio P Basin 1					
Compartment	1	2		3	4
Type	DN-section	Selector section	Anaerobic section 1	Anaerobic section 2	Anaerobic section 3
Volume (m^3)	280	70	230	280	210

The DN-section takes 10 % of the influent flow from the distributer and return sludge flow from the secondary settler. The main purpose of this DN-section is to denitrify the nitrate (NO_3) present in the return sludge with the help of the organic carbon in the wastewater, so that the following selector and anaerobic basins remain fully in anaerobic condition During winter period when the NO_3 concentration in the return sludge increases, in order to maintain the anaerobic condition in the selector, an external carbon source ethanol can be added to the DN-section. The selector section, which is fully anaerobic takes the remaining 80 % of the influent flow from the distributer and also the flow from the DN-section. 80 m^3/h flow, which is about 10% of the total flow is always passing through a trickling filter (No. 8 in Figure 8) via the distributor. Actually there are two trickling filters in parallel connection in this WWTP but only one is in operation. The purpose of passing this small flow is to keep the bacteria alive in the trickling filters, so that they can be operated during high flow conditions. When the flow is more than 1500 m^3/h then the excess flow will pass through the trickling filters via the distributer. It is to be noted that when 80 m^3/h flows is passing to trickling filter the effluent from the trickling filter will be taken to the pumpstation (No 10 in Figure 8) that is just before the aeration basins and then it will undergo biological treatment in the Biodenitro basins (aeration tanks). But when the excess flow will pass through the trickling filters, the effluent from trickling filter will bypass the other treatment steps and is discharged

directly to the Öresund. The area and volume of each trickling filter is 210 m^2 and 887.5 m^3 respectively. The process data of the different compartments of bio P 1 basins are shown in Table 6.

Table 6: Process data of the different compartments in Bio P basin

DN-Section			**Selector Section**					**Anaerobic section (1+2+3)**	
m^3/h	m^3/h	kg/m^3	m^3/h	m^3/h	kg/m^3	kg/day	kg BOD/kg MLSS	m^3/h	min
$(Q+QR)_{max\ normal} = (Q_{dn})_{max\ normal}$	$(Q+QR)_{max\ ATS} = (Q_{dn})_{max\ ATS}$	MLSS concentration	$(Q+Q_{dn})_{max\ normal}$	$(Q+Q_{dn})_{max\ ATS}$	MLSS concentration	Incoming BOD_{design}	Sludge loading	Average daily load	Hydraulic Detention Time
(200 + 1500) = 1700	(300 + 500) = 800	10	(1800 + 1700) = 3500	(2700 + 800) = 3500	4	1900	7	1200	35

Q represents the influent flow
QR represents the Return sludge flow
Q_{dn} represents flow in DN-section

6.4.2.2 Bio P basin 2

The effluent from the bio-P basin 1 will pass through bio P basin 2 for the optimal removal of phosphorus. This basin has total volume of 450 m^3 and has one mixer which will keep the sludge in suspension. The tank is designed for an average flow of 1200 m^3/h and hydraulic detention time of 23 min. In this basin there is also a possibility to add external carbon source when necessary. This dosing of the carbon source is controlled proportionally based on the measured outflow value from the plant. At the end of the bio P basin 2 there is a middle pump station (No. 10 in Figure 8). attached with it. Figure 13 shows the bio P basin 2.

Figure 13: Bio P basin 2

6.4.2.3 Middle pumpstation

The effluent from the Bio P basin 2 will pass to the middle pumpstation and also the effluent from the trickling filters during normal flow condition will be mixed in this pumping station. The reject water from the secondary sludge treatment collected at the reject water tank (No. 18 in figure 8) will also be mixed in this pumping station storage tank. The volume of this tank is 60 m^3 and it contains four pumps. Each pump has a maximum capacity of 900 m^3/h but it can be regulated as needed varying from 360 – 900 m^3/h capacity. This pumpstation has a designed maximum flow of 3500 m^3/h. The wastewater will be lifted up via pumps and will be taken to a influent distributer of the two coupled biodenitro basins/aeration basins.

6.4.2.4 Aeration basin/biodenitro basin

There are two coupled aeration basins also known as biodenitro tanks (No.11 in Figure 8) which are identical and the total volume of both the tanks is 7500 m^3. The two basins can be isolated with two hand operated valves when needed e.g. during some maintenance work. There are 2 influent and 2 effluent weirs in the biodenitro tanks (See Figure 14). The wastewater from the anaerobic basin which has been pumped up to the influent distributer will be directed to either of the two interconnected basins by lowering the influent weir.

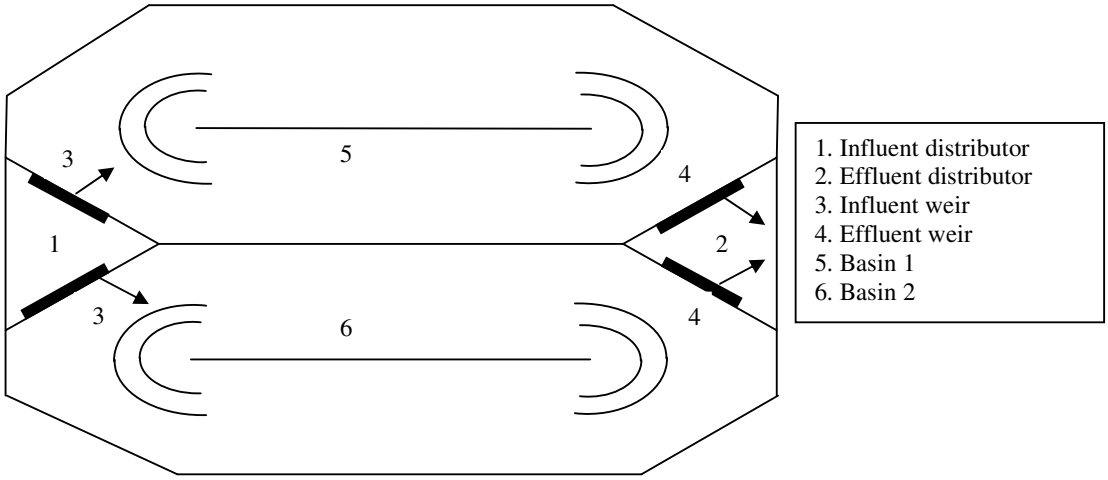

Figure 14: Schematic diagram of two biodenitro tanks (Kruger Oxidation ditch)

The biodenitro tank in which the wastewater will be fed depends on the phases of the different cycles under which the system is in operation. These tanks are operated under biodenipho drift. In the biodenipho process, the tanks are operated under anoxic or aerobic conditions for different time periods depending on the different phases of each cycle. The aerobic and anoxic volumes can be thus changed as needed therefore making the process more flexible. This process provides an opportunity to run these tanks with different sequence of time periods for anoxic and aerobic treatment depending on the time of the day. Hence by this process, the biological reduction of nitrogen can be optimized irrespective of how the waste water composition changes over the day. Each of the biodenitro tanks is equipped with 3 rotors (surface aerators) and 2 mixers (submerged mixer). During nitrification phase (aerobic condition) the aeration is provided by turning on the rotors. Normally by default, the 2 rotors are turned on and have a default oxygen set point of 0-1.5 mg/l O_2. Both the mixers are turned on during anoxic condition to keep the sludge in suspension in a tank. But during aerobic condition mixers are turned on only if one rotor is in operation. Figure 15 shows the effluent weirs of the biodenitro tanks at Lundåkraverket.

Figure 15: Alternating biodenitro tanks showing the two influent weirs

The phosphorus in the wastewater will be reduced in the water phase during both anaerobic and anoxic phase and nitrogen will be removed from water by nitrification followed by denitrification in the form of nitrogen gas. A huge portion of BOD will also be reduced. It is also possible to add dosing with ethanol or methanol if BOD/N ratio is low, that is when the organic material is not enough for denitrification. Dosing with JKL (Iron Chloride) is also possible. The effluent is discharged from any one of the biodenitro tanks and is directed to any one of the secondary settlers depending on the operating cycle.

By the use of ATS-drift (aeration tank settling) operation, it is usually possible to direct all water through the biological step without overloading the post-sedimentation basin during high flow condition. For larger precipitation events ($Q > 2000$ m^3/h) the aeration basin is operated in such a way that some of the sludge is detained without overloading the secondary settlers. At the same time the sludge recirculation will be lowered to about 10%. The maximum design flow of the tank is 3500 m^3/h for both normal and ATS drift.

There is also a room where the online measurements of NH_4-N, NO_3-N and PO_4-P of the wastewater in the aeration tanks are taken. Also, on-spot measurements of SS, oxygen and surface water level are measured on-line. The operation of each single machine is dependent on the time period of the phase program. There are normally 10 phases as default in a cycle that is being operated in Lundåkraverket WWTP. But including the storm water control there are altogether 14 phases set as default in a cycle. A complete cycle of 10 phases (1 to 10) is shown in Figure 16 with default range of time. Four extra phases for storm water control with default range of time is also shown in Figure 17.

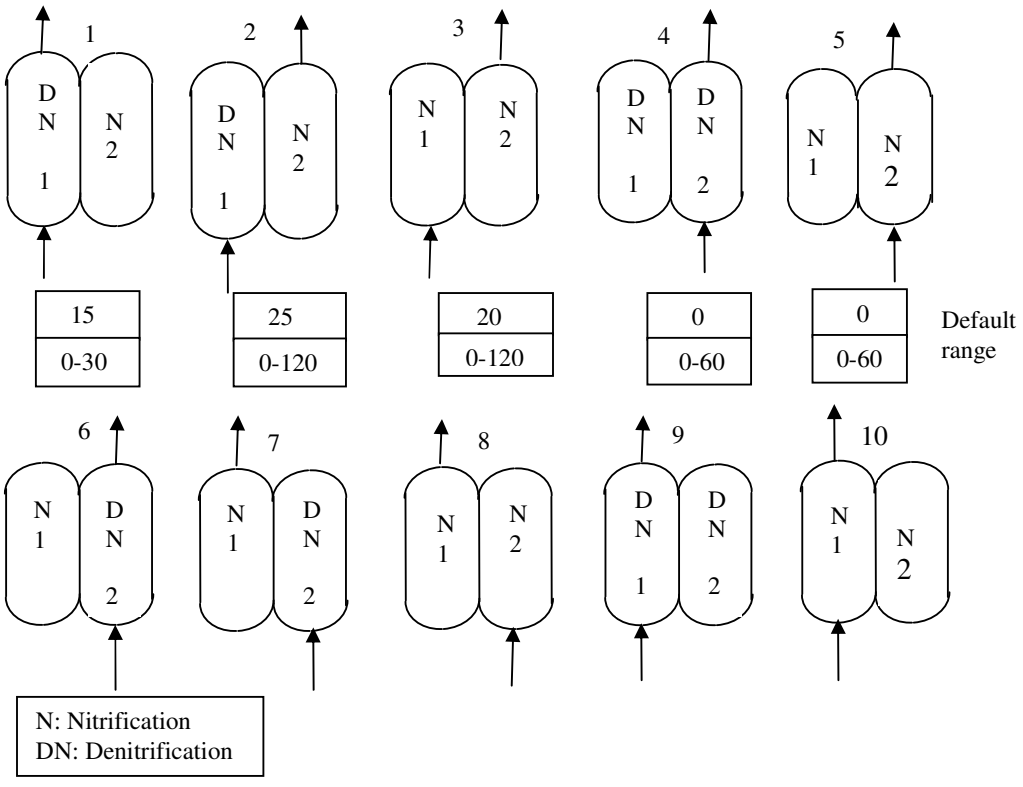

Figure 16: Schematic diagram of a typical 10 phase cycle in Biodenipho-drift

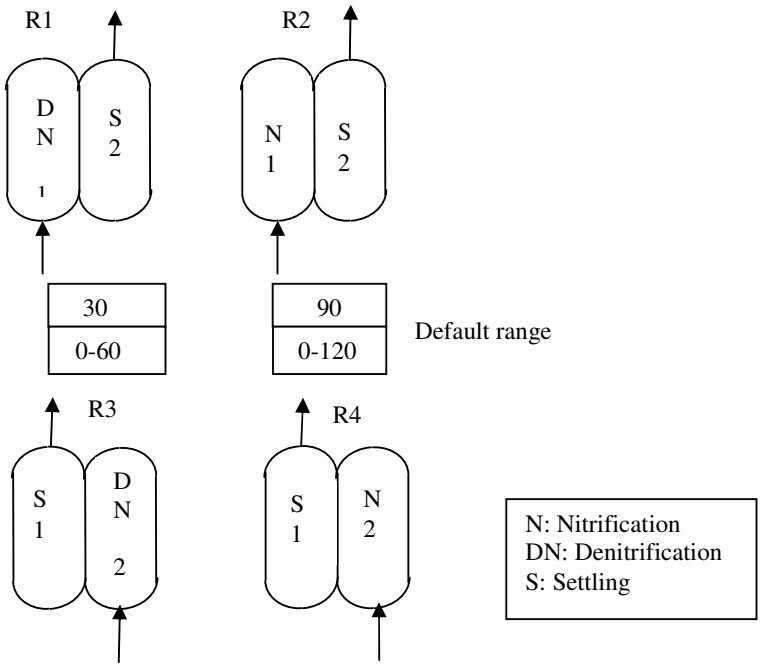

Figure 17: Schematic diagram of the four special phases for storm water control (ATS-drift)

Figure 16 shows that in phase 1, tank 1 acts as denitrification tank/anoxic tank and tank 2 acts as nitrification tank, whereas the wastewater will be fed in tank 1 and the effluent will also go out to secondary settler from the same tank 1 (i.e. denitrification tank here).
In phase 2, tank 1 remains still anoxic for denitrification and tank 2 is also aerobic for nitrification as in phase 1, but the wastewater is coming into tank1 and will exit as effluent from tank 2 to secondary settlers.
In phase 3 the direction of flow will remain same as in phase 2 but both the tanks will be aerobic for nitrification.
In phase 4, tank 2 will be fed with wastewater from anaerobic basin and will exit as effluent from same tank whereas both the tank will be in anoxic condition for denitrification.
In phase 5, the direction of flow is same as in phase 4 that is influent will be taken to tank 2 and effluent will be taken out from same tank 2 but both tank will be in aerobic condition for nitrification.
The phases 6, 7, 8, 9 and 10 are the mirror images of phases 1, 2, 3, 4 and 5 respectively.

Figure 17 represents the four extra phases for the storm drift: R_1, R_2, $R_3$and R_4. Phases R_3 and R_4 are the mirror images of phases R_1 and R_2 respectively.

Duration of 10 phases in eight different cycles is shown in Table 7.

Table 7: Phase duration in the different cycles

Cycle	1 (min)	2 (min)	3 (min)	4 (min)	5 (min)	6 (min)	7 (min)	8 (min)	Stormwater operation (min)
Phase 1, 6	18	18	10	0	0	0	0	0	18
Phase 2, 7	72	62 m	62	63	54	36	18	0	62
Phase 3, 8	0	10	18	27	36	54	72	90	10
Phase 4, 9	0	0	0	0	0	0	0	0	0
Phase 5, 10	0	0	0	0	0	0	0	0	0
DN (%)	50	45	40	35	30	20	10	0	Phase 7

From the Table 7 it is seen that the denitrification percentage varies from 0 to 50% depending on cycle in operation. It is also seen that all 8 cycles have only either 4 phases or 6 phases in operation and the ATS drift has 6 phases in operation. Each cycle consists of a 3 hours time period. The nitrification can be increased during winter by increasing the aerobic phases to 90%.

Table 8 shows the daily operating cycles in Lundåkraverket WWTP. Only cycles 2, 4, and 5 are in operation and the DO set point varies from 0.9 to 1.5 mg/l. Comparing Table 7 and Table 8 it can be assumed that during 00:00 to 09:00 hours mainly cycle 2 is in operation where denitrification period is 45 %. This implies a low-flow condition with a reduction of aeration time (nitrification percent). Similarly, during 18:00 to 24:00 hours it is operating in cycle 5. Comparing with Table 7 it can also be seen that denitrification period is 30 %, which may be due to high loading of wastewater during that period of time. Similarly it can be seen from Table 8, the different cycle operating during different period of time in a day of a week.

Table 8: Daily operating cycles at Lundåkraverket WWTP

Hours	00:00-03:00		03:00-06:00		06:00-09:00		09:00-12:00		12:00-15:00		15:00-18:00		18:00-21:00		21:00-24:00	
	Cycle	DO	Cycle	DO	Cycle	DO	Cycle	DO	Cycle	DO	Cycle	DO	Cycle	DO	Cycle	DO
Monday	2	1.0	2	0.9	2	0.9	4	1.2	4	1.5	4	1.2	5	1.2	5	1.2
Tuesday	2	1.0	2	0.9	2	0.9	4	1.2	4	1.5	4	1.2	5	1.2	5	1.2
Wednesday	2	1.0	2	0.9	2	0.9	4	1.2	4	1.5	4	1.2	5	1.2	5	1.2
Thursday	2	1.0	2	0.9	2	0.9	4	1.2	4	1.5	4	1.2	5	1.2	5	1.2
Friday	2	1.0	2	0.9	2	0.9	4	1.2	4	1.5	4	1.2	5	1.2	5	1.2
Saturday	2	1.0	2	0.9	2	0.9	4	1.2	4	1.5	4	1.2	5	1.2	5	1.2
Sunday	2	1.0	2	0.9	2	0.9	4	1.2	4	1.5	4	1.2	5	1.2	5	1.2

6.4.2.5 Secondary settler/ secondary sedimentation basin

The effluent from the aeration tanks will pass to secondary settler (No. 12 in Figure 8) where suspended particles or flocs are allowed to settle. There are two secondary settlers in Lundåkraverket WWTP which are in parallel connection and are circular in shape. There is a periphery running sludge scrapper in each of the settlers which will direct the sludge to the sludge pit in the middle of the basin. From the central sludge pit the sludge is carried through a weir to a return activated sludge tank (No.18 in Figure 8) where the flow from each basin is measured. Return sludge flow is controlled by an automatic overflow weir in order to maintain the desired flow. The total (combined) area and volume of both the secondary sedimentary tanks is 1700 m^2 and 6000 m^3 respectively, each basin having a diameter of 33 m with an average depth is 3.5 m. The sludge from bottom of the tank will be taken as return sludge to pre-denitrification basin (DN-section of Bio P basin 1) via the return activated sludge tank. During normal flow condition the return sludge will be 30-100% of the total flow with the default value set at 70 %. The maximum return sludge flow is 1000-2000 m^3/h with a default value of 1500 m^3/h and the minimum return sludge flow varies from100-500 m^3/h with a default value of 500 m^3/h. A small portion of the sludge is also removed as excess sludge (waste sludge/bio-sludge) which will undergo sludge treatment. The secondary settlers have the maximum design flow of 3500 m^3/h. A picture of a secondary settler at Lundåkraverket is shown in Figure 18. There is also a facility of online measurement of sludge and flow of return sludge in the treatment plant. Finally, the effluent from the secondary settler will undergo final treatment (chemical treatment) via a bypass chamber (No. 13 in Figure 8).

Figure 18: A Secondary settler in Lundåkraverket WWTP

6.4.3 Final treatment / chemical treatment

The final treatment reduces specially the suspended materials and phosphorus. Lundåkraverket WWTP consists of following final treatment units:

- Flocculation basin
- Lamella sedimentation

It should be mentioned here that there is also a bypass chamber before the chemical treatment (No. 13 in Figure 8). It has a capacity of 3000 m^3/h whereas the chemical treatment units have a capacity of only 2500 m^3/h. Therefore, the main function of this chamber is to bypass the excess flow coming out of the secondary settlers to receiving water.

6.4.3.1 Flocculation basin

The volume of flocculation basin is 625 m^3 and the design flow is 2500 m^3/h. There are six flocculation basins and polymers and chemicals are added to form flocs of suspended particles where phosphorus gets attached to it. At Lundåkraverket, only a small quantity of $Fe_2(SO_4)_3$ and a major part of $Al_2 (SO_4)_3$ is added to form flocs.

6.4.3.2 Lamella sedimentation basin

There are three lamella sedimentation basins with a total area of 3125 m^2 and the design flow is 2500 m^3/h. The effluents from the flocculation basin are allowed to settle in a lamella sedimentation basin. The lamella plates are 2.5 m deep and the sludge gets attached to these plates which skids down and is collected at the bottom. From the bottom of this basin, the chemical sludge is removed and taken for sludge treatment and the effluent is discharged to Öresund harbour.

6.5 Observed phosphorus peaks at Lundåkraverket

Occasional phosphorus peaks have been observed in the effluent at Lundåkraverket WWTP. From the available data for periods between 1997 to 2006 it has been observed that the most severe conditions (high P peaks) were recorded between the years 2002 and 2004 with effluent P concentrations as high as 5.1 mg/l. This is shown in Figure 19. The peaks normally lasted for a week before it started appearing again after 3-4 weeks but sometimes it appeared more frequently. After 2005 these peaks started to appear less frequently. Figure 20 shows a one day online measurement variation of the effluent concentrations. The concentration that has crossed the x- axis limit in the figure represents the P concentration. Concerned authorities at Lundåkraverket WWTP are trying hard to find a solution to this problem but with little success.

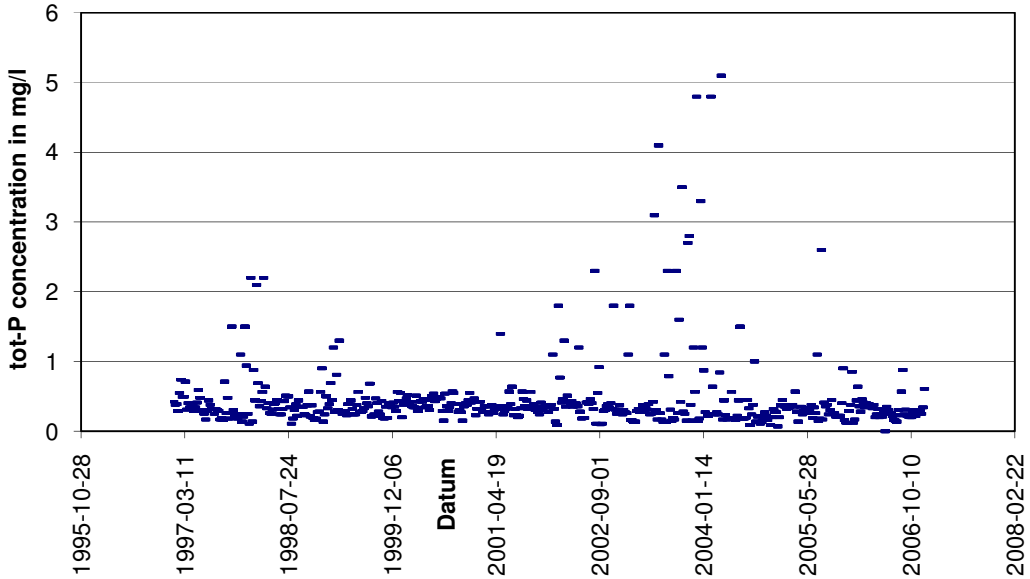

Figure 19: Effluent P concentrations at Lundåkraverket WWTP during 1997-2006

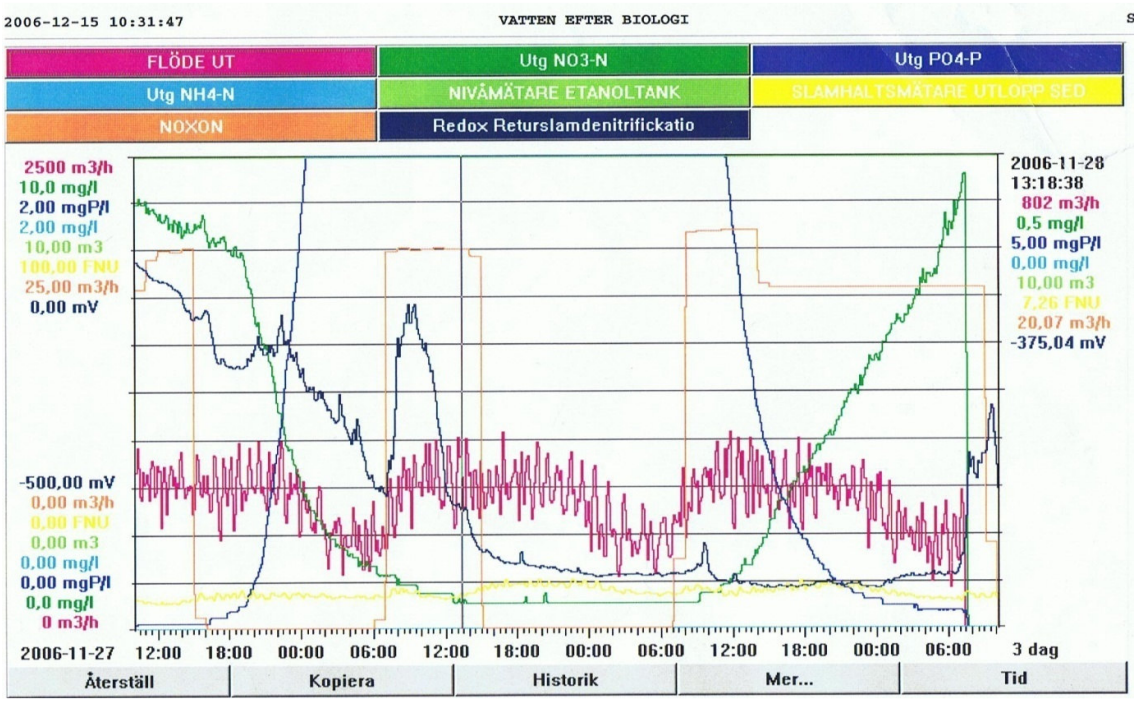

Figure 20: Online meter showing effluent phosphorus peaks measured on 2006.12.14 (source: Lundåkraverket WWTP)

7.0 Computer modelling

An effort has been made to describe and simulate the different operational conditions in the Lundåkraverket WWTP with the help of computer modelling. Computer modelling is one of the best available solutions today to simulate the actual process conditions in a WWTP. With day to day advancement in technology, new computer software have been developed with more and more additional features and advanced options which make the user easier to deal even with the most complicated problems encountered in the actual treatment plant. It is not always possible to simulate everything that is going on in the treatment. Depending on the available information on the WWTP , appropriate data, efficiency and user skills, the range of options varies. Here, the computer program EFOR has been used as a tool for computer modelling.

7.1 A brief introduction to EFOR

EFOR is a computer model available from the DHI Water and Environment group which can be used quite effectively to simulate many different operating conditions in a WWTP with ease and comfort due to its user friendliness and flexibility. It is based on well known mathematical models used for biological treatment in the activated sludge processes such as ASM-1, ASM-2, CNDP. (EFOR, 2003).

The best thing about EFOR is that it is divided into four very distinct parts: Plant, Operation, Data and Simulation. In the Plant section it is very easy to create or edit the physical structure of the treatment plant very quickly and accurately. There are different types of treatment plants to choose from by default, such as the simple activated sludge treatment plant, biodenitro, biodenipho etc. EFOR also allows the user to create a treatment plant from scratch which is however recommended only for advanced users.

In the Operations section it is possible to operate the WWTP with the help of different control loops with various sensors and control devices such as pumps, valves etc in the WWTP units. It is also possible to choose the different phases and cycles in this section.

The Data section deals with all kinds of inlet characteristics such as the incoming wastewater composition, process temperatures etc.

Finally, in the Simulations section, model is run if all the required information has been provided correctly in the previous sections.

Another great feature of EFOR is the chart and table manager where there are options to view the results and simulations either in tables or graphs quite easily. In short, EFOR is a very well designed computer model with a lot of simple as well as advanced features which are beyond the scope of describing here in this short introduction. For more detailed information please refer to the EFOR 2003 user guide.

7.2 Description of model components

7.2.1 Introduction

The type of configuration used in this model is the biodenipho configuration which is also one of the available default models in EFOR. It should be mentioned here that the part of the

treatment plant that deals with sludge handling and treatment is omitted in this model. Mainly the mechanical and the biological treatment up to the secondary clarifier are considered whereas the final/chemical treatment is not included.

Two different sets of models had to be used in this study. As previously mentioned this study was conducted based on the data from 1997 to 2006. The rejectwater coming from the secondary sludge was added just before the primary settlers at Lundåkraverket WWTP till 2004. After 2004, this rejectwater was removed and added just before the aeration basin. Due to this change, two different models were constructed indicating the rejectwater as an external dosing in the model. Figure 20 represents the second model with the external dosing added just before the aeration basins and data used for this model is from 2005 to 2006. More importance is given to model 2 as this reflects the present condition at Lundåkraverket WWTP. An important thing to be noted is that data from 2002 to 2004 has not been considered in this study due to lack of sufficient information required to run the model efficiently.

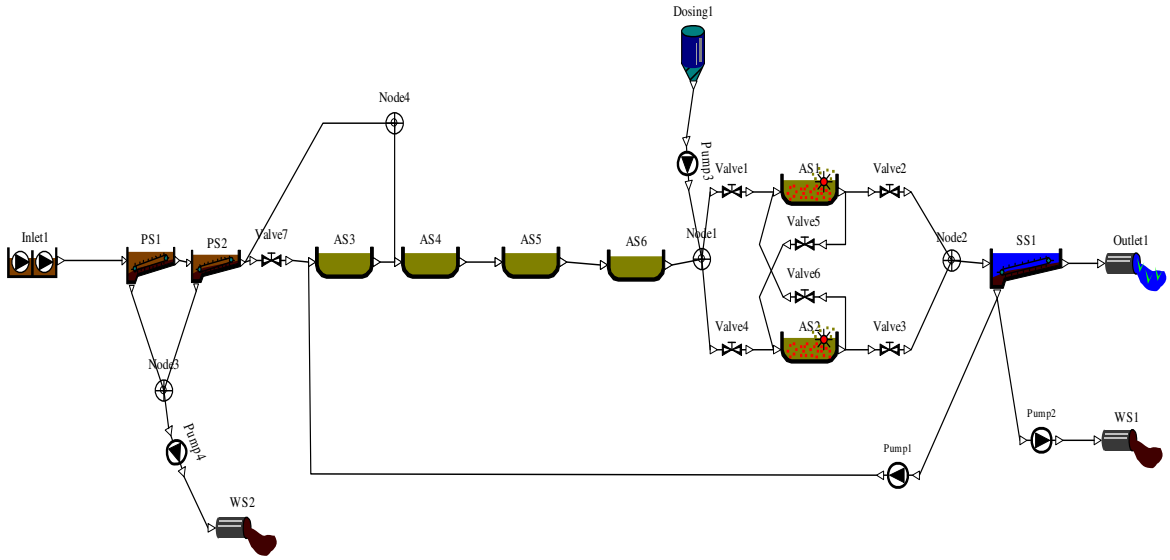

Figure 20: Different components of the treatment plant used in model 2

Abbreviations used in Figure 20:
AS: Activated sludge
PS: Primary sedimentation
SS: Secondary sedimentation
WS: Waste sludge

Study carried out on model 1 is incomplete. This is due to the fact that during the middle of this study being conducted, it was informed that the addition of rejectwater from the secondary sludge were not coming in before the primary settlers anymore, but it was removed and was added before the aeration basins. Therefore keeping in mind about the present situation, continuing working with model 1 seemed meaningless and therefore a new model had to be built from scratch with the changes in external dosing and was named model 2. Model 2 was chosen to be the main model. Calibration of model 1 is not shown here. Please refer to Appendix B to view the calibration.

7.2.2 Definition of the different components

As seen in the figure, there are two primary settlers - PS1 and PS2. Each Primary settler has four compartments in the actual treatment plant, but in the model these compartments have been omitted. Dosing 1 represents the rejectwater from secondary sludge which is connected to node 1 through pump 3. WS2 represents the waste sludge from the primary settlers PS1 and PS2 which will undergo sludge treatment and are connected through node 3 and pump 4.

In the actual treatment plant there are only two anaerobic tanks – Bio P basin 1 and Bio P basin 2. Bio P basin 1 is divided into four chambers and bio-P basin 2 has no division. The bio P basin 1 configuration is shown in section 6.4.2, Table 5. For simplicity in the models, the DN-section is indicated by AS3, selector section is indicated by AS4 and the anaerobic section is denoted by AS5. The anaerobic section 2 and 3 shown in Table 5 is represented by a single tank in the model which is AS5. AS6 represents the bio P basin 2.

AS1 and AS2 are the two biodenitro tanks which are connected by different valves: valve1, valve2, valve3, valve4, valve5 and valve6.

SS1 represents the secondary settler in the models. In reality, there are two secondary settlers in parallel connection with the biodenitro basins. But in the model they have been combined into a single tank for simplicity.

The line connecting SS1 to AS3 represents the return sludge and pump 1 is used to control this activity.

In the model, a major portion (90%) of the wastewater coming out from the primary settler will pass to the tank AS4 and 10% of the flow will pass to tank AS3. But in the actual case, approximately 80% of incoming flow from the primary settlers will pass to the selector section and 10% will pass to the trickling filters and another 10% will finally pass to the DN-section. It should be mentioned here that the trickling filters are not taken into account in the model because they are mainly used during high flow conditions in the WWTP and also the required effluent data for the trickling filters are not available. Valve7 is used to control the flow volume or in other words to maintain the 10 % flow into AS3. Introduction of node 4 is to observe the concentrations of the analytical components from PS1.

7.3 Definition of model parameters

7.3.1 Total average flow rate

According to the data provided from Lundåkraverket WWTP (personal conversation with Jan-Erik Petterson), a total average incoming flow of 600 m^3/h has been used in model 1 and 700 m^3/h in model 2. In reality, the flowrate is not constant but varying with time but here an approximate mean flow rate is used.

7.3.2 Incoming wastewater composition

The actual wastewater composition in a WWTP is always changing depending on water usage and the waste generated by human activities as well as the industries. But in the model, the average annual values have been chosen as the basis for the inlet characteristics and also parameters for modelling. These parameters are: BOD_7, COD, Tot-P, Tot-N, NH_4-N and SS. Please refer to Section 6.2, Table 3 for the incoming wastewater composition.

7.3.3 Process temperature

Biological processes are mostly temperature dependent and also temperature is not always constant. It is changing depending on the warm summers and cold winters. The growth rate and survival of organisms in an activated sludge is to a large extent affected by temperature changes. Due to this reason it is vital that an appropriate temperature be provided in the models. From the supplied data, a constant temperature of 15°C has been chosen for model 1 and 20°C is chosen for model 2.

7.3.4 Area and volume of WWTP units

As mentioned in section 12.2.2, some modifications have been made in the model compared to the existing WWTP units in Lundåkraverket for simplification. The areas and volumes used in the model are shown in Table 9.

Table 9: Dimensions of the WWTP units used in model

Unit	Area (m^2)	Depth (m)	Volume (m^3)
PS1	320	1.56	Not applicable in the model
PS2	570	1.85	,,
AS3	Not applicable in the model	2	280
AS4	,,	2	70
AS5	,,	2	720
AS6	,,	4	450
AS1	,,	4	3750
AS2	,,	4	3750
SS1	1700	3.53	Not applicable in the model

7.4 WWTP operation

In EFOR, control loops are used to operate the WWTP with the help of different sensors, controllers, control devices and set points. Therefore, it is important that correct devices are attached to or linked to get the optimum results. (EFOR, 2003).

7.4.1 Definition of the different pump capacities and their operation

There are altogether four pumps that are attached to the pipes of the different WWTP units concerned. The maximum flow that the pump can generate is characterised by the pumping capacity (EFOR, 2003). The pump capacities used in this model is shown in Table 10.

Table 10: Pump capacities

Pumps	Connecting units	Capacity (m^3/hr)
Pump 1	SS1-AS3	1000
Pump 2	SS1-WS1	10
Pump 3	Dosing1-Node1	7.5
Pump 4	Node3-WS2	2.5

Pump1 is used to control the return sludge flow. It is set to operate in a one phase cycle with the help of a percentual controller with direct operation which indicates that it will be operated to reflect the actual wastewater flow in the return sludge. The set point here is set to 70% which indicates that the return flow will be 70% of the inlet flow as the sensor used here is "flow in inlet 1".

Pump 2 controls the flow between the secondary settler and the waste sludge 1. This capacity is an assumption that has been made from the given data that a large portion of flow is going through the return sludge and only a small portion flows to the waste sludge 1. It is set to operate in a one phase cycle with an on/off controller with direct operation. The direct form indicates that it is set to be activated when it exceeds the set point. Type of sensor used is SS in AS1. This type of controller is used to activate/deactivate a control device whenever it reaches a certain set point (EFOR, 2003). In order to control the sludge concentration it is common to use a set point in the range of 4000-5000 gSS/m^3 (EFOR, 2003). In this model a set point of 4500 gSS/m^3 is used. By using a set point of c with a pump capacity of 10 m^3/h in the model, the desired average waste sludge flow of around 230 m^3/d at Lundåkraverket WWTP is maintained.

Pump 3 is used to control the flow from the external dosing. Type of controller is on/off-direct which is set to operate in a one phase cycle.

Pump 4 is used to control the flow of primary sludge from PS1 and PS2. The pump capacity reflects the approximate average flow of 60 m^3/h from the primary sludge in the WWTP. The type of controller used here is also an on/off controller with direct activity. The sensor used here is the flow from inlet 1 and the set point is set at 2.5 m^3/h. It is set to run at a single phase.

7.4.2 Definition of control loops in the aeration basins AS1 and AS2

Surface aerators/rotors have been used here as oxygen source. AS1 and AS2 are the two aeration chambers that form a part of the biodenipho configuration. As this an aerated chamber the sensor used here is oxygen. The type of controller is Step-Inverse. A step controller activates the control device starting from zero and thereafter increases in steps to the maximum capacity of the control device. After reaching the maximum value, its capacity will decrease step-wise again. The Inverse function is used to activate the control device when the measurements are below the set points (EFOR, 2003). In the model the step is set to 10% of the maximum value.

From section 6.4.2, Table 8, it has been observed that in Lundåkraverket WWTP, cycle 4 is mainly in operation most of the time. So, this particular cycle has been used in the model to reflect the real situation. Also from Table 7, it can be seen that cycle 4 is a four phase cycle with phase 2, 3, 7 and 8. Phase 2 and phase 7 has a time period of 63 minutes and phase 3 and phase 8 has a time period of 36 minutes. Again referring to Section 6.4.2, Figure 16, the flow pattern of this typical four phase cycle with phases 2, 3, 7 and 8 can be observed. So, in order to reflect similar flow pattern, the governing phase length, aeration time, and valve operations in the models have been represented in Table 11.

Table 11: A four phase cycle showing the governing phase length, DO set-point and valve operations

Name	Phase1	Phase2	Phase3	Phase 4	Unit
Phase length	63	27	63	27	minutes
Aeration AS1	0	1.5	1.5	1.5	g/m^3
Aeration AS2	1.5	1.5	0	1.5	g/m^3
Valve1	100	100	0	0	% open
Valve2	0	0	100	100	% open
Valve3	100	100	0	0	% open
Valve4	0	0	100	100	% open
Valve5	100	100	0	0	% open
Valve6	0	0	100	100	% open

7.6 Calibration of model 2

In reality, the influent wastewater composition of the WWTP is always varying. But in the model a constant yearly average value has been assigned. Therefore modelled effluent values from the different units in the WWTP show some deviation from the reality. As a result some changes had to be made to the model parameters in order to bring the model close to the reality. This calibration can be made by changing the model parameters based on mass balances of the different fractions of inlet wastewater characteristics, model processes and conversions.

7.6.1 Calibration of primary settlers

After assigning all the required parameters to the model, simulations were started. After the first simulations the concentrations of the analytical components from the primary settlers and final effluent were observed. There was no information regarding the effluent concentrations from the primary settlers at Lundåkraverket WWTP. Based on the information that 50-70 % of SS will be removed in primary settler as mentioned in Section 3.2.1, the concentration of SS from the effluent of the primary settler in the model is calibrated to bring approximately 65 % removal of SS. Table 12 shows the effluent SS concentrations before and after calibration.

Table 12: SS concentrations from the primary settlers before and after calibration

Incoming SS concentration in the primary settler (mg/l)	221
Assumed effluent SS concentration from the primary settler in WWTP (based on 65 % SS removal) (mg/l)	77.35
Observed SS concentration from primary settler before calibration (mg/l)	89.40
Observed SS concentration from primary settler after calibration (mg/l)	77.21

As seen from Table 7.4, the modelled primary settler effluent SS is higher than the observed values at Lundåkraverket WWTP. Therefore, few default model parameters had to be changed in order to bring the modelled effluent concentrations of SS from primary settler close to the one that has been assumed to be the observed concentrations in the treatment

plant. Table 13 shows the changes that have been made to the default model values for PS calibration.

Table 13: Changes in default value for PS calibration

Parameter	Symbol	Default value	Changed value
Maximum reduction of SS (%)	KMax	75	80
Constant for SS dependency (g/m^3)	Kss	50	42

By simple trial and error method, a 80 % reduction of SS and a constant for SS dependency of 42 g/m^3 will give a 65 % reduction of SS in the primary settler.

7.5.2 Calibration of the final effluent concentrations

Since the effluent concentrations from biological treatment is not available, the final effluent concentrations from the model is compared with that of the final effluent from the WWTP. But it has to be mentioned here that the model comprises only the biological treatment section whereas the chemical treatment has been completely omitted. But the available final effluent data from the WWTP represents the concentrations after going through both biological and chemical treatment. Hence the modeller is compelled to calibrate the final effluent from the model with that of the final effluent from the treatment plant. From this it can be said that calibration of the model has made it more efficient than the actual treatment plant.

Table 14 represents the changes in the various parameters that were observed in the model effluent before and after calibration with that of the observed value in the WWTP.

Table 14: Changes observed before and after calibration

Parameters	Observed effluent (mg/l)	Modelled effluent before calibration (mg/l)	Modelled effluent after calibration (mg/l)
COD	<30	34.08	17.8
N_{tot}	13.307	10.11	13.82
NH_4	3.99	1.46	3.84
$NO3 + NO_2$	9.71	7.20	9.32
SS	4.45	33.62	4.47
P_{tot}	0.346	4.06	0.39

The simulated final effluent is slightly different from the observed values. This variation in the results could be either because the modelled effluent values are from biological treatment only whereas the observed effluent values were recorded after going through both the biological and the final chemical treatments. However, in order to get an effluent quality in the model somewhat similar to the WWTP, the model was calibrated by changing some default parameters in the model. The changes noted in Table 14 are also shown in Figure 21.

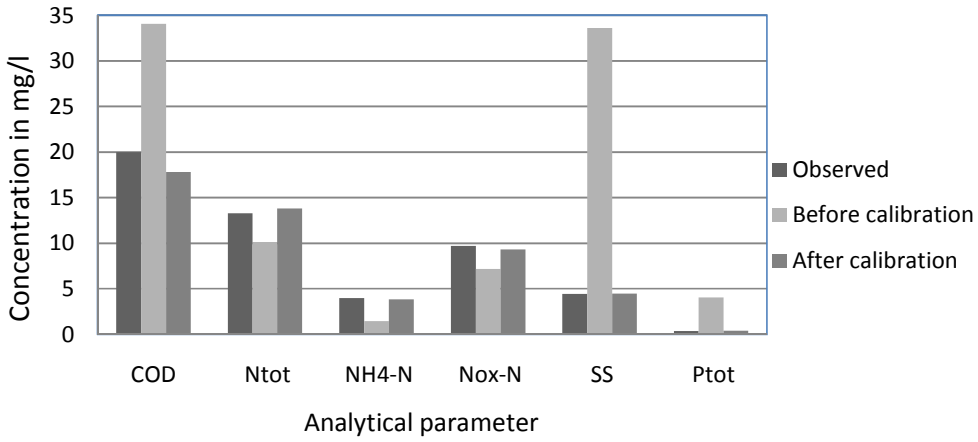

Figure 21: Observed and simulated final effluents

Table 15 shows the parameters that has been changed for the calibration so that model will reflect observed final effluent in the WWTP.

Table 15: Changes in default values for calibration of the final effluent

Name	Symbol	Default value	Changed value	Unit
Fraction of nitrogen in inert COD	fni	0.030	0.040	gN/gCOD
P content in biomass	fpb	0.015	0.010	gP/gCOD
Decay rate of heterotrophs	Bh	0.320	0.720	1/d
Fraction of denitrifiers	etag	0.600	0.470	
Saturation for nitrate	Knoh	0.500	0.250	gN/m^3
Maximum growth rate of autotrophs	Mua	0.900	0.860	1/d
Decay rate of autotrophs	Ba	0.150	0.300	gN/m^3
Saturation for ammonium	Kna	0.200	0.200	gN/m^3
Maximum growth rate of polyP biomass	Mup	1.00	2.50	1/d
Decay rate of polyP biomass	Bp	0.200	0.050	1/d
Saturation for P	Kp	0.200	0.100	gP/gCOD
Minimum concentration of non-setteleable SS	SSinit	4.00	1.00	gSS/m^3
Maximum settling rate	V0	8.90	16.0	m/hr
Sludge characterization 1	n1	1.50	0.47	m^3/mg

From Figure 21 it can be clearly observed that phosphorus and SS concentrations in the model before calibration is much higher than the observed values in the WWTP. Also, the nitrate plus nitrite concentrations in simulated effluent is lower than observed values. So, in order to increase the nitrate plus nitrite concentrations the decay rates of heterotrophs have been increased from its default value as shown in Table 15. Similarly, the fraction of denitrifiers and saturation of nitrate were decreased to raise the nitrate plus nitrite values in the model.

In the same way, to lower P concentrations from the modelled effluent, the maximum growth rate and the decay rate of poly-P biomass were increased and decreased respectively as shown in Table 15.

The values of total nitrogen and ammonium were adjusted by changing the parameters, maximum growth rate of autotrophs, decay rate of autotrophs and saturation for ammonium.

Likewise, minimum concentration of non-settleable SS was reduced to 1.0 gSS/m^3 in the model to increase setteleable fraction of SS. Finally, the sludge characterisation 1 and the maximum settling rate is adjusted to 0.47 m^3/mg and 16 m/hr respectively.

8.0 Results and discussions

In order to find possible explanations regarding the occasional phosphorus peaks in the effluent, a number of process conditions responsible for the deterioration of bio P process were simulated with the help of EFOR. These simulated results were tried to be analysed with the help of knowledge acquired from the literatures. The different conditions that were tried to check the P peaks in the effluent is described below.

8.1 Effect of diluted wastewater

High flows during low loaded conditions cause dilution of the incoming wastewater. This kind of condition is likely to occur mainly during a high rain event for a prolonged period in a low-loaded condition such as weekends. This dilution can have an adverse effect on the effluent concentrations due to deficiency of available substrates which ultimately impacts the bio P efficiency of the WWTP.

In order to check the effects of a diluted wastewater in the model, the model was run for 24 hours with a 22hour initial dilution period and the remaining 2 hours with a normal loaded period. The effect of this condition on the phosphorus effluent in the model is represented by simulation day 1 in Figure 22. Similarly, the model was run for 2, 3, 4, 5, 6 and 7 days in order to check the impact of the 22 hour dilution period in the coming 6 days having a normal loaded condition. In the model this kind of dilution period was achieved by increasing the incoming wastewater flow and decreasing the loading of COD, BOD, phosphorus and acetate concentrations in the influent. The effects of this diluted wastewater periods were simulated and results were plotted.

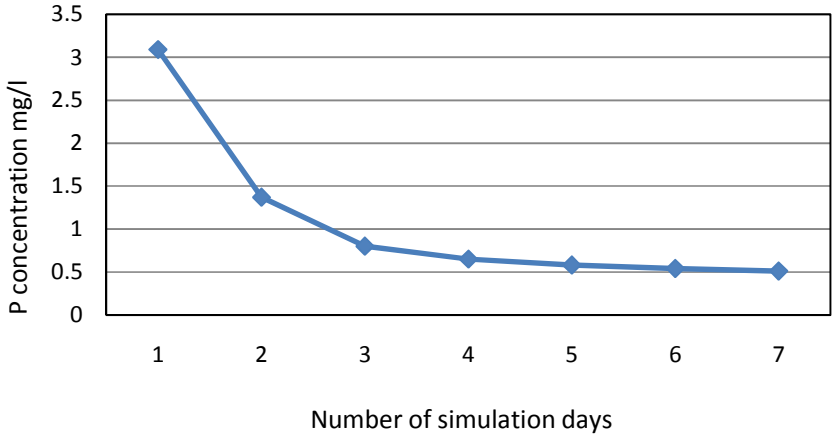

Figure 22: Effect of a 22 hour dilution period on P peak

From Figure 22 it can be seen that the phosphorus concentration on the outlet was almost 8 times (3.2 mg/l) higher than the normal effluent values (0.39 mg/l) on the 1st day of the simulation. This peak gradually decreased with time and on the 7th day of simulation the P concentration was 0.5 mg/l.

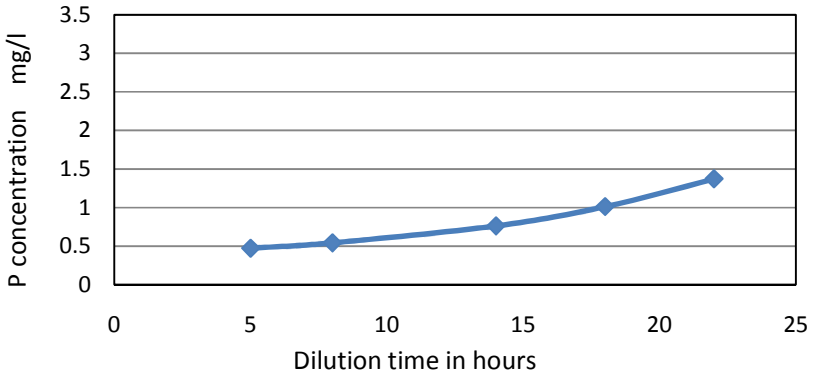

Figure 23: P concentration with 5, 8, 14, 18 and 22 hour's dilution period for a 2 day simulation

In the next part a dilution period of 5, 8, 14, and 22 hours were studied for a 2 days time period. This was done by running the model for an initial 5 hour dilution period and the remaining 42 hours with a normally loaded period. The dilution period was subsequently increased from 5 hours to 22 hours and its impact on the effluent P peaks were observed. Figure 23 clearly illustrates that longer the incoming wastewater dilution period, the more will be the P peaks at the outlets. After 2 days simulation, the outlet value of the P peak for a dilution period of 5 hours is 0.47 mg/l whereas for a 22 hours dilution period the P concentration was observed to be 1.37 mg/l.

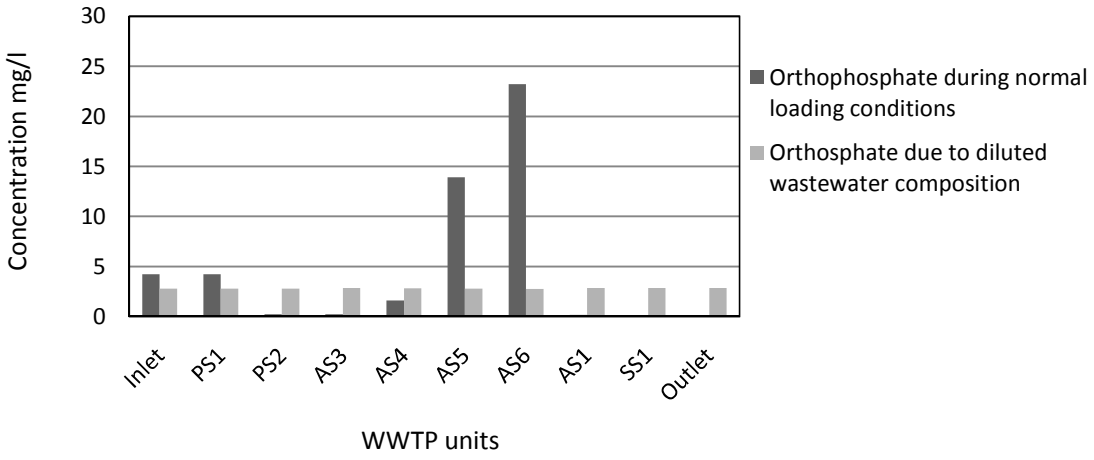

Figure 24: Variation in orthophosphate concentrations in different units during normally loaded and 22 hours of dilution period with a 2 days simulation

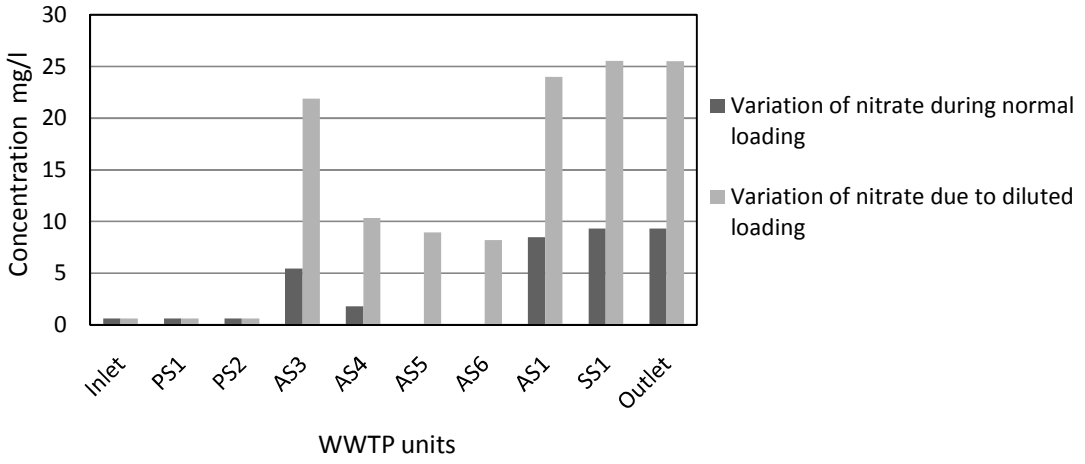

Figure 25: Variation of nitrate concentrations in the WWTP units during normally loaded and 22 hours of dilution period with a 2 days simulation

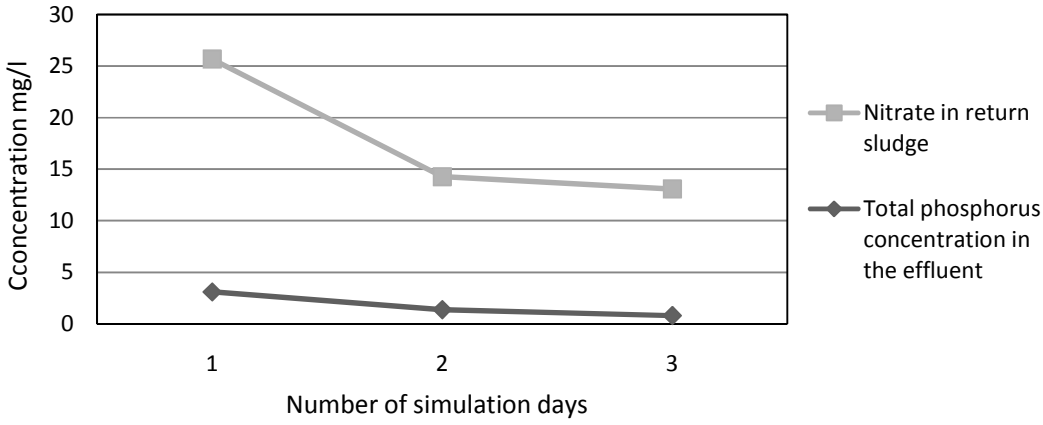

Figure 26: Effect of 22 hours dilution period of incoming wastewater in the effluent P concentration and nitrate in the return sludge in different simulated days

From Figure 24 it is noticed that with a 2 days simulation, the orthophosphate concentration during the normally loaded condition is being gradually released in the anaerobic tanks AS4 to AS6 and after that there is a P uptake taking place in one of the biodenitro tank AS1. But in the diluted condition the orthophosphate concentration is almost constant. This deterioration of the bio P process leading to a high P concentration at the outlet could be due to lack of enough substrate for denitrification. This causes a high nitrate concentration in the outlet as well as in the return sludge which further worsens the bio P efficiency as the anaerobic tanks are not fully anaerobic or is in an anoxic state as shown in figure 25. The presence of nitrate in the anaerobic tanks prevent P release (orthophosphate release) in the anaerobic tanks. Figure 26 illustrates the proportional variation of nitrate in return sludge and phosphorus in the outlet.

Dynamic variations of orthophosphate concentration in one of the biodenitro tanks AS1 during the 2 days simulation for a dilution period of 22 hours is presented in Figure 27.

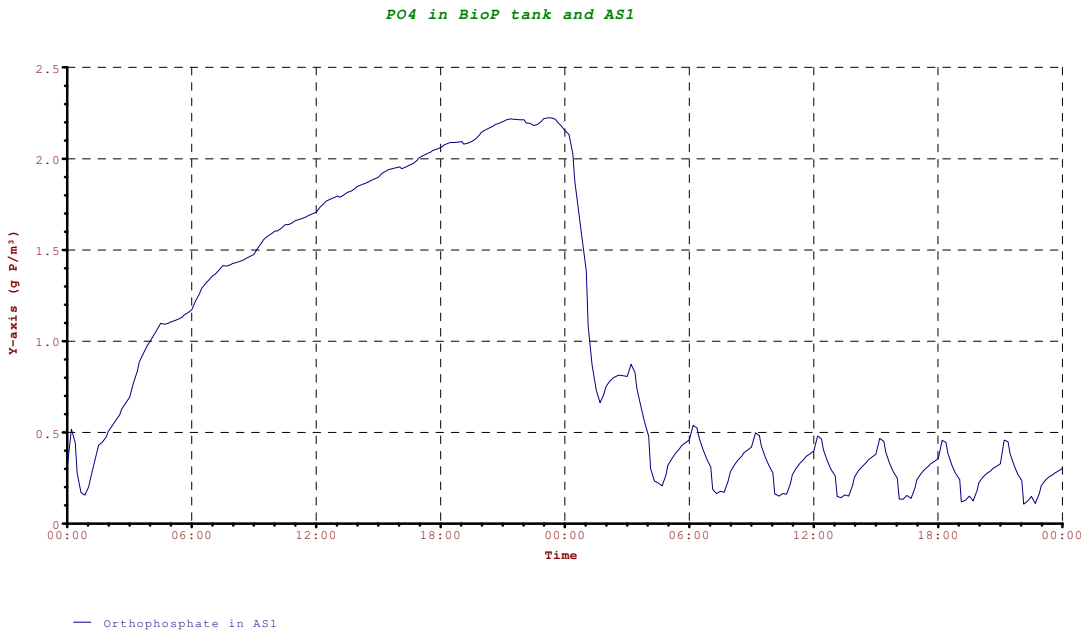

Figure 27: Dynamic variation of orthophosphate in AS1 with 2 days simulations and 22 hours dilution period

From Figure 27 it can be seen that in AS1, P uptake is taking place only after 24 hours. The absence of a P uptake during the first 24 hours reflects the effect of the dilution period where P uptake is deteriorated. This may be due to the fact that less substrate is available for the bio P bacteria to store it as PHB which influenced the P uptake in the aerobic tank AS1. But as the flow normalises back the p-release takes place instantly but P-uptake needs time to fully recover. Hence, the high P concentration in the effluent as shown in Figure 22 and Figure 23 may be due to the fact that P release takes place instantly when influent flow returns to normal strength. But P uptake will not fully recover unless PHB storage of PAOs is achieved as put forward by Termink *et al*. (1996).

8.2 Effects of low flow condition

A low-flow condition in a WWTP can occur when the incoming flow is reduced during nightly flow. This situation may hamper the bio P efficiency due to lack of fresh nutrients and may cause VFA uptake in the anaerobic zone and subsequently less P uptake in the aerobic zones. There is also a possibility of P release in the secondary settlers due to a long retention time of the sludge if the settler is oversized.

The effect of low flow condition were studied by reducing the incoming wastewater flow from 700 m^3/hr to 300 m^3/hr for a duration of 5 hours and 10 hours. The result of this change is presented in table 16.

Table 16: Effect of low flow condition

Duration of low flow condition (hrs)	Simulation days (d)	Total P concentration at the outlet (mg/l)	Sludge age (d-hrs)
5	1	0.44	7-20
5	2	0.40	7-09
10	1	0.44	9-02
10	2	0.42	7-21

From Table 16 it is observed that the P concentrations have slightly increased with an incoming low loading of 5 and 10 hours but is still within the outlet limit of 0.5 mg/l at Lundåkraverket. From this table it is also noticed that even after a low flow condition of 10 hours the sludge age didn't show any drastic increase.

Figure 28 shows the dynamic variation of the sludge blanket in the secondary settler for a one day simulation for a 5 hour initial low-flow condition. It is seen that there is no significant variation in the sludge blanket due to the low-flow condition.

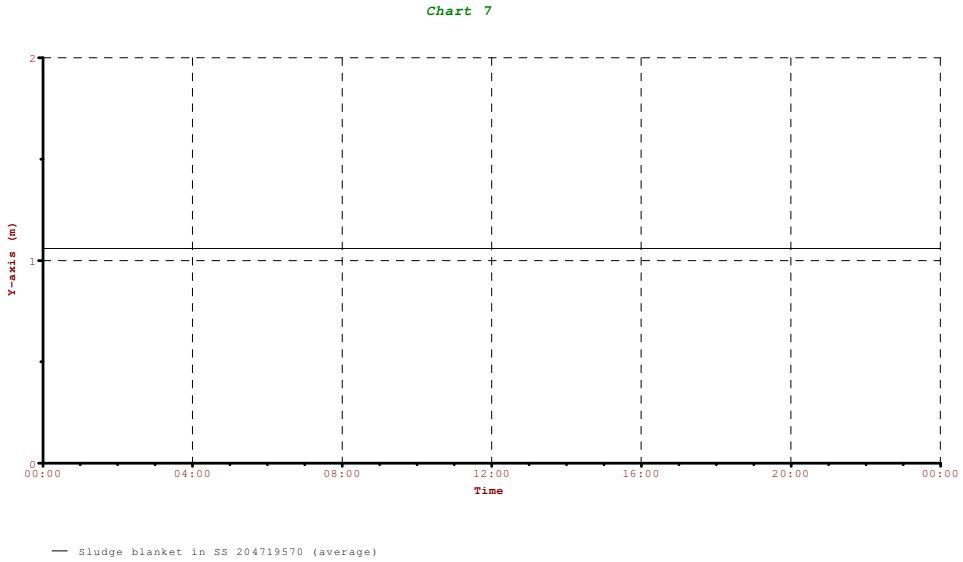

Figure 28: Dynamic variation showing the sludge blanket in the secondary settler for low-flow condition

8.3 Temperature effects

In order to check the temperature and pH effects on the effluent concentrations, first of all the incoming wastewater temperature was changed from the current value of 20°C to 15°C and 10°C respectively for a three day period and the corresponding P concentrations were noted. It was found that there was no significant increase in the P values from the current ones. When the temperature was decreased from 20°C to 15°C the P concentration only slightly increased from 0.39 mg/l to 0.42 mg/l. Similarly, further decreasing the temperature to 10°C, P concentrations increased to 0.47 mg/l. No further decrease in temperature was performed because at Lundåkraverket the minimum temperature recorded was approximately 10°C.

8.4 High acetate in the wastewater composition

Acetate is favourable for PAOs as well as GAOs. When wastewater contain acetate, it may affect the bio P process because the GAOs use these valuable substrate in anaerobic zone but do not contribute to P uptake in aerobic/ anoxic zones.

An external dosing with a high acetate dosing was introduced just before the primary settlers to check its effects on the effluent P concentrations. Doing so, it was observed that the P concentration decreased from 0.39 mg/l to 0.17 mg/l. Further increase in the acetate concentration did not show any deterioration of bio P but it enhanced the bio P efficiency perhaps due to the readily available substrate such as acetate.

8.5 Operational problem with the return sludge pump

An operational problem that could have occurred in the return sludge pump that stopped the pump for few hours was studied with the help of computer modelling. This type of problem may cause P release in the secondary settler due to high sludge retention time in the settler. It was achieved in the model by changing the operation of the return sludge pump by introducing a 3 phase cycle instead of the normal one phase cycle. In a 72 hour cycle, the first phase was assigned a duration of 48 hours, the second phase with a duration 6 hours and the third phase with a duration of 18 hours. The pump was operated with a return sludge flow of 70 % of the incoming flow for the 1^{st} and the 3^{rd} phase. On the other hand in the 2^{nd} phase the set point was set to zero in order to stop the pump for six hours. After running the simulations, the effluent values of phosphorus concentrations did not show any alteration. But while observing one of the aeration tanks AS1 during the dynamic simulations, it was noticed that orthophosphate, SS and nitrate concentrations showed a sharp deviation at that particular time when the pump was turned off. But the curve eventually smoothed off in the next phase. In Figure 29, the dynamic variation of orthophosphate in one of the biodenitro tanks AS1where a slight increase in orthophosphate level was noticed just after the pump was stopped for 6 hours. This may be due to the decrease of poly P biomass in the AS1 tank caused by stopping the return sludge pump.

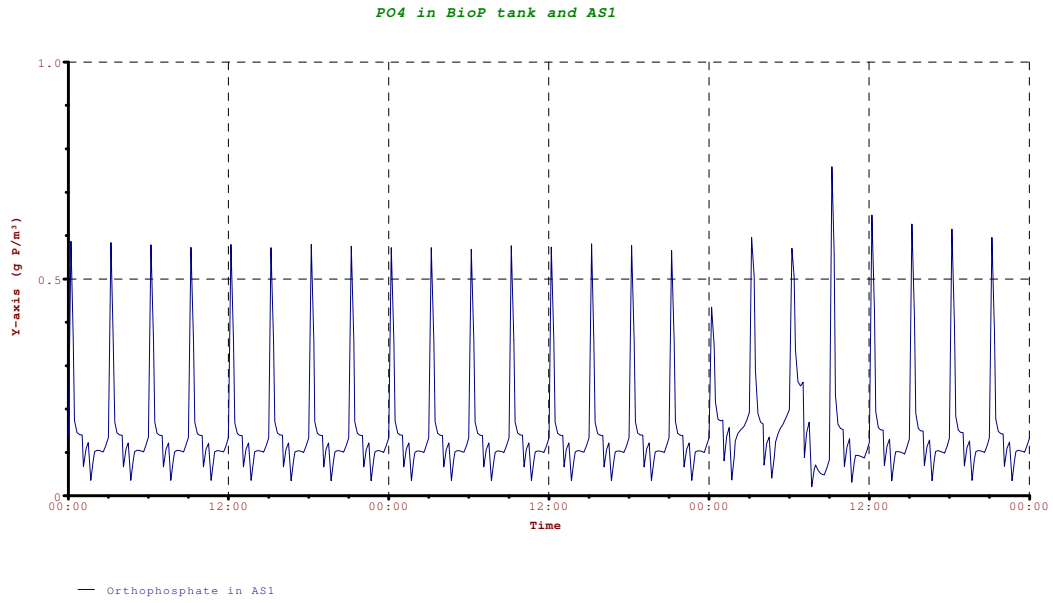

Figure 29: Variation of orthophosphate in AS1

From Figure 30, in the dynamic variation of SS in AS1, a depression of SS concentration is noticed during the period when the pump was stopped.

Figure 30: Variation of SS in secondary settlers

Figure 31 shows a sharp increase in the sludge blanket during the stopping of pump for 6 hours. This sudden change smoothed out with time and was normalised. Due to this phenomena no P peaks was observed in the effluent however.

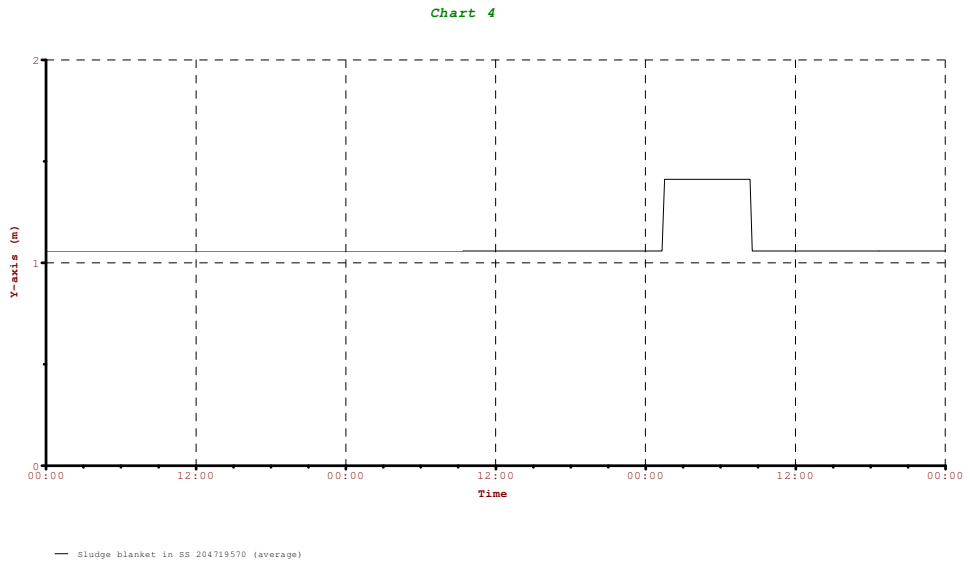

Figure 31: Dynamic variation of sludge blanket in secondary settler after stopping the return sludge pump

8.6 Effect of trickling filters

The effect of trickling filters couldn't be modelled in this study due to lack of sufficient information about its effluent parameters. There is a possibility that during high flow conditions when more effluent from the trickling filter is added before the aeration basin, more nitrate and phosphorus is added to the aeration basins. This may worsen the bio P efficiency during the dilution period in two ways:

First, when available substrate is less, the added nitrate from the trickling filters cannot be properly denitrified. This will increase nitrate in the return sludge stream and if this nitrate is not denitrified due to the lack of substrate, the anaerobic tanks will be in anoxic states. Therefore, no orthophosphate release will take place and also storage of PHB in the poly P biomass will not take place. As a result P uptake will not take place in the aerobic/anoxic tanks.

Second, when the effluent phosphorus from the trickling filters is added to the aeration basin. This effluent does not possess any bio P bacteria that store VFA as an energy source for P uptake in the aeration basins. This can lead to high P concentrations in the effluents.

9.0 Conclusions

The occasional phosphorus peaks encountered at Lundåkraverket WWTP used to last for about a week. From this study it can be concluded that the diluted incoming wastewater with a longer period of time can increase the effluent phosphorus peaks that gradually decreases in about a week. This kind of situation is likely to occur during low-flow weekend conditions with the dilution caused by prolonged rainfall.

During the dilution period there will be deficiency of substrate for denitrification which will cause an impact on the anaerobic P release and subsequent aerobic P uptake. P release is deteriorated due to lack of readily degradable substrate and the presence of nitrate in the anaerobic tanks from the return sludge stream.

A low-flow condition for a maximum period of 10 hours doesn't show any P peaks. So, there is less probability of occurrence of the P peaks in the effluent at Lundåkraverket WWTP due to low-flow condition.

EFOR is a helpful tool for studying the different process conditions in a WWTP. But the accuracy of the results vary depending upon available data required for proper calibration, amount of time spent on the model as well as a thorough knowledge about the WWTP operations. Considerable amount of patience is also a must during the calibration process because a small change in a particular parameter influences some other parameters as well, thereby causing lot of confusion during the early stages of modelling.

Because of the highly versatile living conditions of *Microthrix parcivella*, it dominates most of the other filamentous organisms and therefore can survive in a number of difficult circumstances. It is due to this fact that the main operational cause of its existence in WWTPs is very difficult to ascertain. Temperature is supposedly one of the most important factors effecting its growth among other factors. From the knowledge gathered from a variety of literature studies, a number of control strategies have been put forward. These strategies were found effective in a number of WWTPs depending on the intensity and duration of the problem.

10.0 Recommendations

The following recommendations have been put forward by the authors based on their study on Lundåkraverket WWTP:

To find out a more reasonable solution for the problem associated at Lundåkraverket WWTP, it would be wise to collect more detailed information about the WWTP as well as carry out different tests to check the bio P efficiency in a more thorough manner.

Tests such as P release and P uptake tests should be conducted in order to know the exact condition of the phosphate removal capabilities in the activated sludge.

Separate batch experiments may be performed where possible, to find out the sludge process rates such as maximum denitrification rates, nitrification rates, oxygen uptake rates etc.

At Lundåkraverket WWTP, many important parameters such as the exact incoming wastewater flows, effluent from primary and secondary settlers, exact COD values from effluent etc are missing. It would be a good idea to keep a record of these parameters regularly and therefore try to maintain a more efficient database system that will be beneficial for future studies as well as for maintenance purposes.

It is also recommended to try to remove the addition of effluent from the trickling filters for a limited period of time and check if it helps to prevent the phosphorus peaks in the final effluent.

It would be a good idea to add a dosing of propionate just before the anaerobic basin during high P concentrations at the outlet. This will probably reduce the P peaks because of the extra substrate.

EFOR is a good modelling tool for simulating WWTP process conditions. But there are some unknown bugs that create frustrating problems in the middle of an ongoing simulation without any prior warning or recommended solutions. Sometimes even the result files disappear without any known apparent reason. Therefore, additional research on EFOR is expected to find possible solutions to these unsolved problems.

Most of the supplied papers related to the different process conditions at Lundåkraverket WWTP were in Swedish language, which was one of the most frustrating part for the authors because of their very limited Swedish language skills. Therefore, it would be sensible to provide such information in English language wherever possible.

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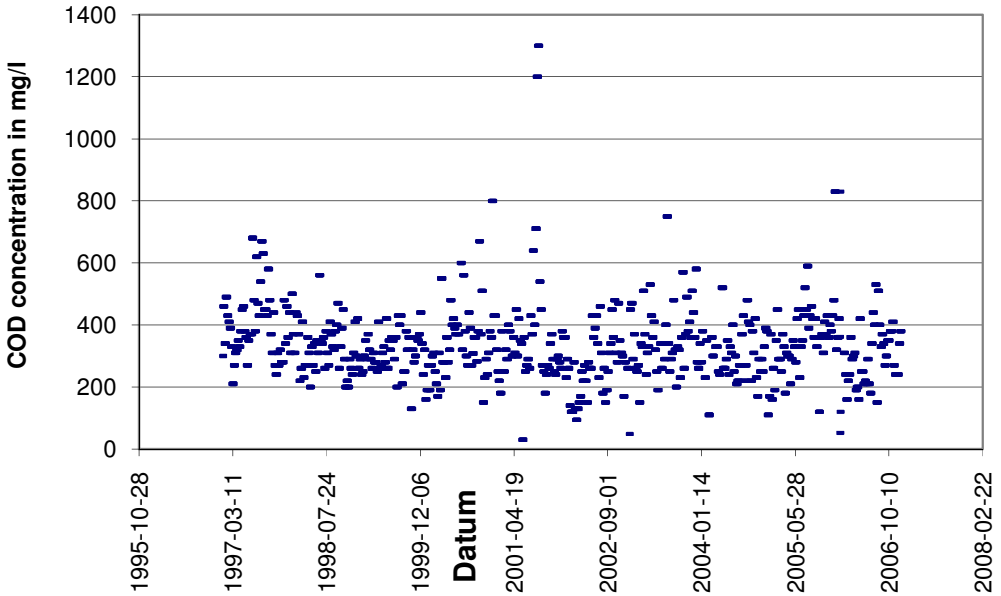
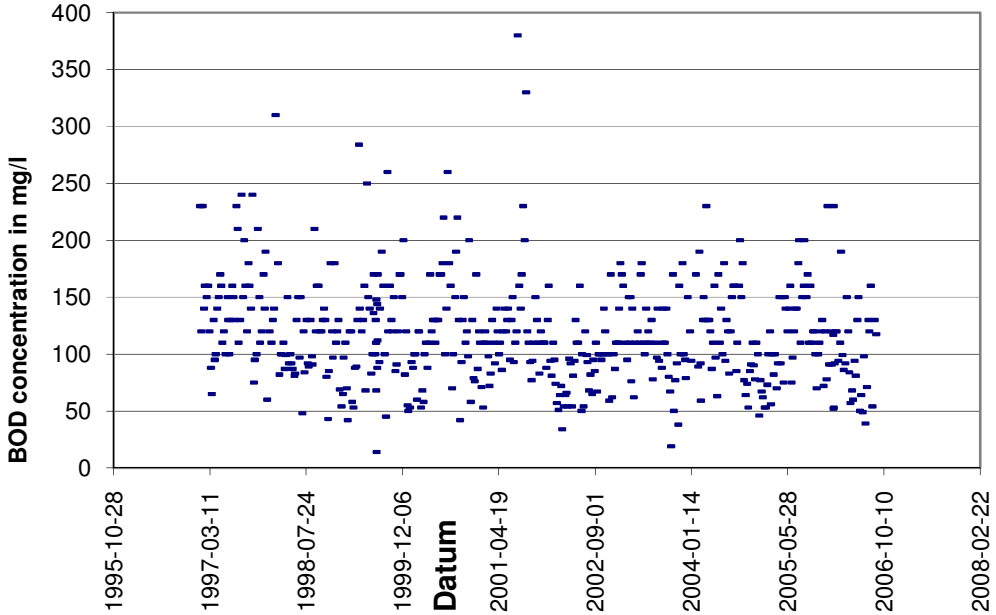
Appendices

Appendix A: Daily variations of incoming COD, BOD, phosphorus, nitrogen, ammonium, SS and temperature

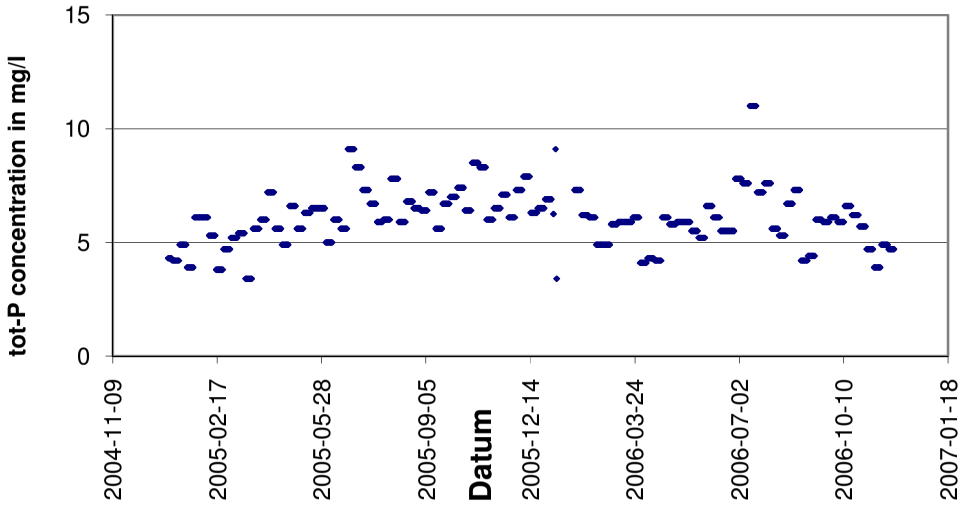
A. Graph showing incoming COD variations at Lundåkraverket WWTP



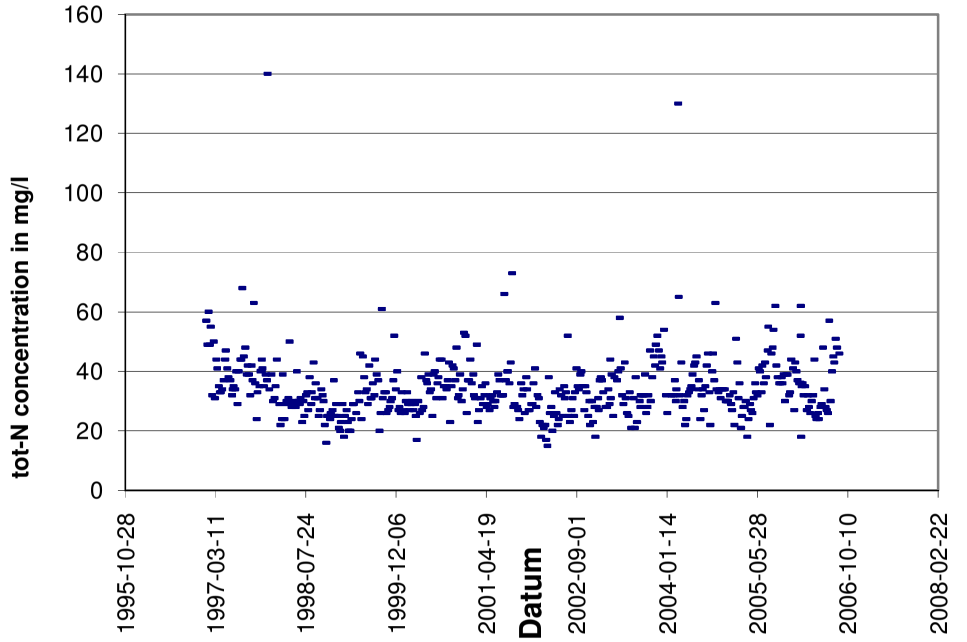
B. Graph showing incoming BOD variations at Lundåkraverket WWTP



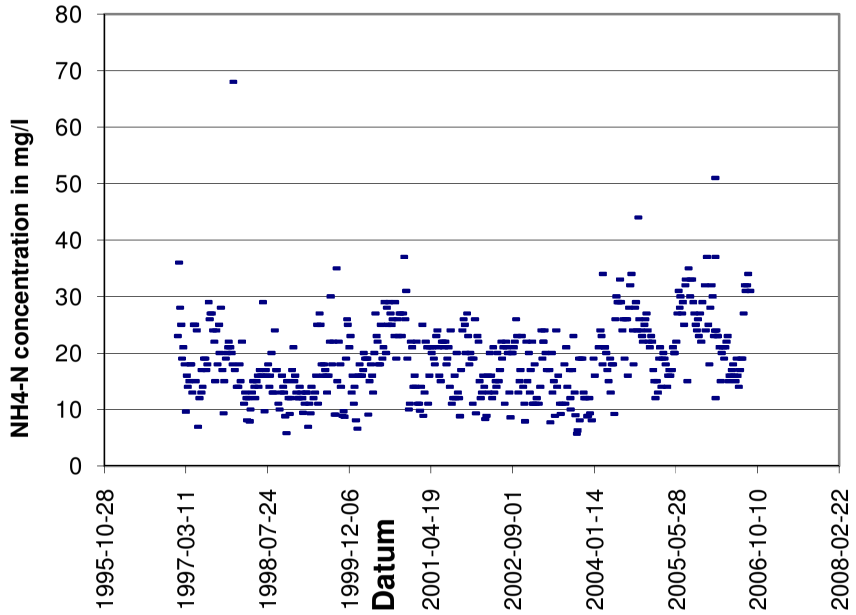
C. Graph showing incoming P variations at Lundåkraverket WWTP



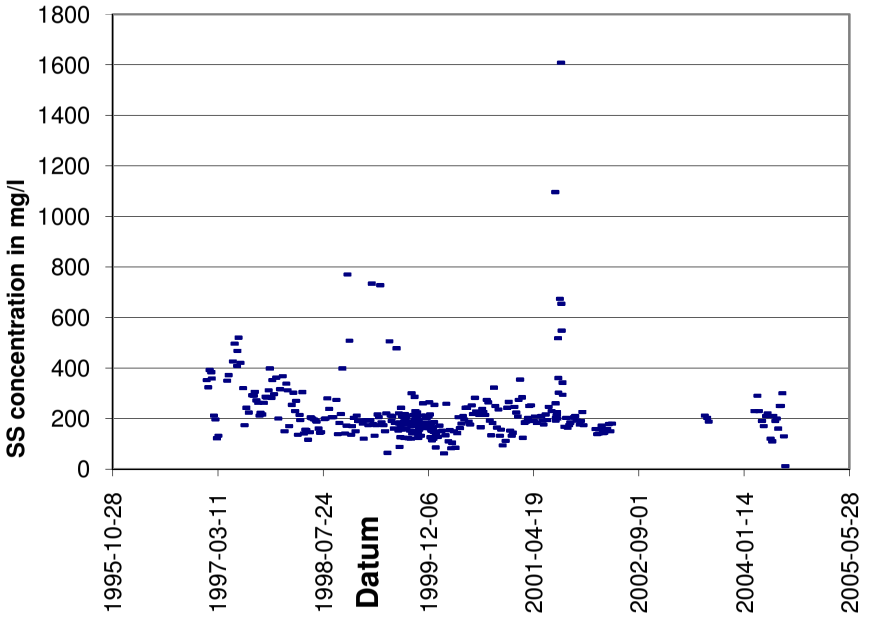
D. Graph showing the incoming total N concentrations at Lundåkraverket WWTP



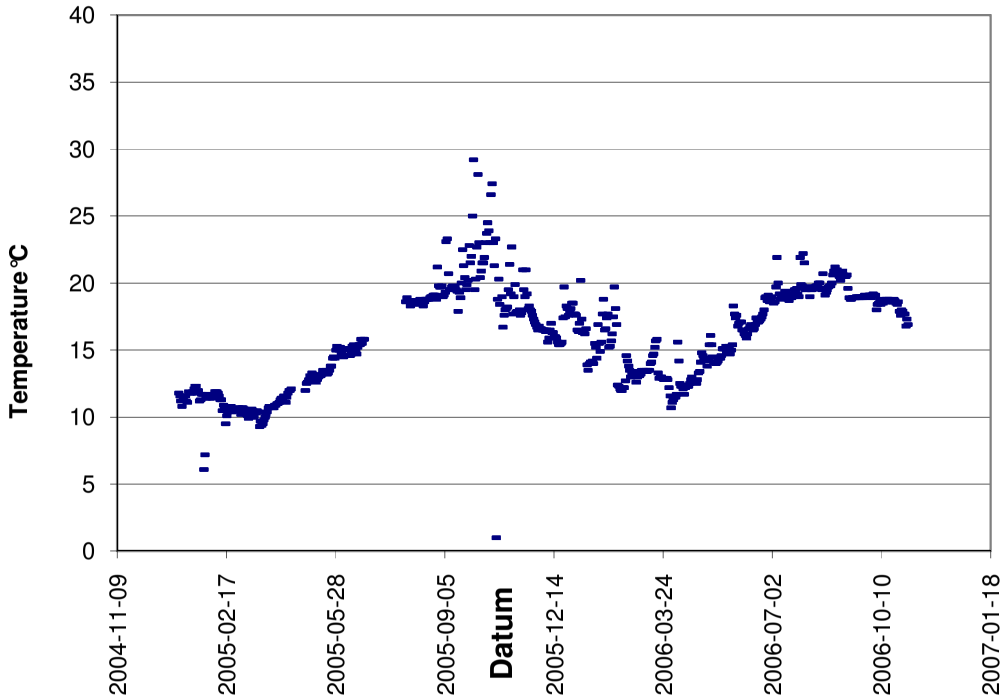
E. Graph showing the incoming NH4-N concentrations at Lundåkraverket WWTP



F. Graph showing the incoming SS concentrations at Lundåkraverket WWTP



G. Graph showing the incoming temperature variation between 2005- 2006



Appendix B: Calibration of model 1

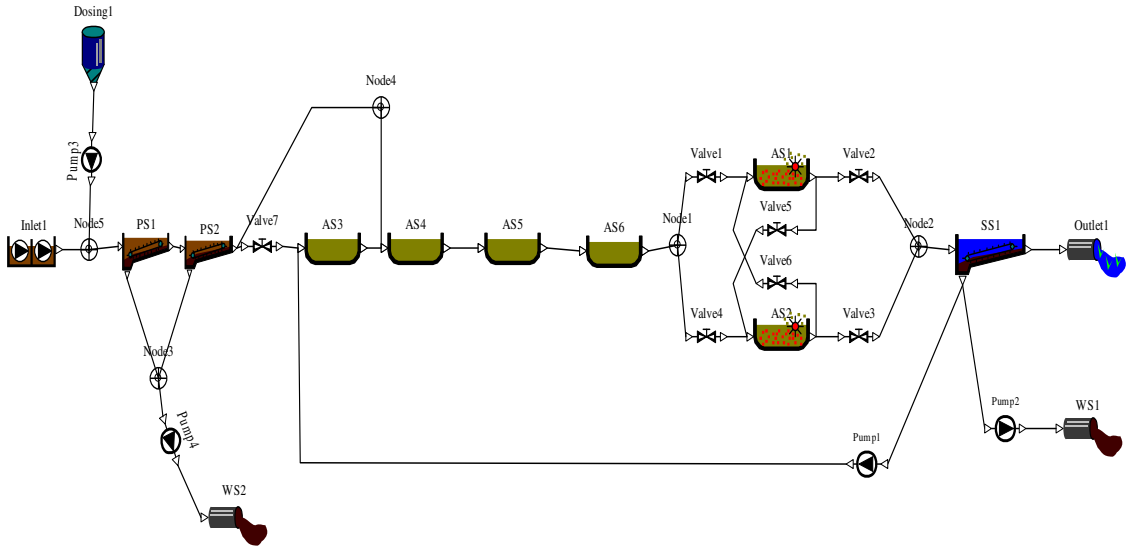


Figure B1: Different components of the treatment plant used in model 1

Before calibration of model 1, it is important to state that all the definitions of model parameters are explained under section 7.0 during the definition of model 2. So here only the changes in the data that were made in model 1 from that of model 2 are stated. The main difference in model 1 from model 2 is the location of rejectwater addition. In model 1, the rejectwater from the secondary sludge is added just before the primary sedimentation settler 1 (PS1) as shown in FigureB1. Other important changes are the influent flow rate, influent quality and pump capacity. This model was actually calibrated to reflect the process condition of WWTP before 2004 when the reject water was coming in before primary sedimentation basin which is indicated as dosing in model 2. The available yearly average data from year 1997-2001 were taken as inputs for influent data and calibration. The influent data for model 1 is shown in Table B1.

Table B1: Yearly average values of incoming wastewater data from 1997-2001 at Lundåkraverket WWTP

Flow (m^3/hr)	**Temp (°C)**	**BOD_7 (mg/l)**	**CODt (mg/l)**	**Total P (mg/l)**	**Total N (mg/l)**	**NO3-N (mg/l)**	**NH4-N (mg/l)**	**SS (mg/l)**
600	15	126	350	6.0	34.0	1.9	18	225

The pump capacity that had been assigned for model 1 is shown in Table B2.

Table B2: Pump Capacity of different pumps in model 1

Pumps	Connecting units	Capacity (m^3/hr)
Pump 1	SS1-AS3	1000
Pump 2	SS1-WS1	10
Pump 3	Dosing1-Node5	4.5
Pump 4	Node3-WS2	2.5

The pump capacity was assigned to work at full capacity to bring the same average flow as that in Lundåkraverket during the year 1997-2001. Pump 4 which is a return sludge pump is set to operate at 51 % of inlet flow and type of controller is percentual. This represents that flow in the pump will be proportional to inlet flow. The set point is set at 51 % in the model to bring the same average flow as that in the WWTP. All the other parameters and flow scheme and operation are same as that in model 1 as explained in section 7.0. After assigning all the required parameters to the model, simulations were started. After simulation it was observed that the simulated effluents are slightly varying than the effluent from the primary sedimentation basin and final effluent at Lundåkraverket WWTP. So in order to make the model reflect the real condition at WWTP, the model has to be calibrated so that the effluent value from the model resembles to that of WWTP.

Calibration of primary settlers (PS)

The variation in different effluent parameter from the primary settler of WWTP and the model before calibration are shown in Table B3.

Table B3: Variation in effluent parameters from primary settler before and after calibration

Concentration	CODt (mg/l)	SS (mg/l)	Total N (mg/l)	NH4-N (mg/l)	NO_x-N (mg/l)	Total P (mg/l)	Total P (filtrated) (mg/l)
Observed effluent from PS at WWTP	219.6	85.575	28.678	22.147	3.342	4.638	3.032
Simulated effluent from PS before calibration	227.61	140.29	32.40	24.46	1.88	5.37	4.86
Simulated effluent from PS after calibration	220.07	86.61	29.9	22.84	3.31	4.31	3.42

Due to differences in the model effluent values from that of the WWTP, some of the parameters had to be changed for calibration. Primary settlers are capable to remove only the particulate materials, so the fraction of particulate COD (CODx) and soluble COD (CODs) and fraction of particulate phosphorus (TPx) and soluble phosphorus (TPs) were altered from

the default value to calibrate the PS. Also the fraction of readily degradable fraction (fss) was changed from default value. To reduce the total COD and SS, the CODx value was increased to 215 (mg/l) from the default value 210 (mg/l). Also the fraction of readily degradable COD fraction in particulate COD (fss) was changed to 0.120 (gCOD/gCOD) from default 0.10 (gCOD/gCOD). To bring the model effluent total phosphorus and filtrated phosphorus the value of TPx was changed from1.20 to 2.63 mg/l and the value of TPs changed from 4.80 to 3.37 mg/l. NOx-N in the inlet data was changed to 3.34 from 1.90 as the effluent value of NOx-N has been increased in effluent due to the addition of rejectwater. Also the reject water has been diluted to 65 %. After these changes the primary settler is more or less calibrated to reflect the effluent quality closer to the effluent from the primary settler of WWTP. The effluent value from the model after the calibration is shown in Table C3. After this calibration the final effluent from the model is calibrated to bring the value close to that of the effluent from the secondary settler in WWTP.

Calibration of secondary settlers

In order to calibrate the secondary settlers in the model to bring it closer to the effluent from secondary settler in WWTP, few model default parameters were changed. Some of the model default parameter were changed to calibrate the final effluent and these changes is presented in Table B4 below. The minimum concentration of non-settleable solids and sludge characterization1 was also changed to 3.5 gSS/m^3 and 1.1 m^3/mg from its default value of 4.0 gSS/m^3 and 1.5 m^3/mg respectively. The maximum settling rate (V0) is also increased to 12 m/hr from its default value 8.9 m/hr in reduce suspended solids in the effluent.

Table B4: Changes in default model parameter

Name	Symbol	Default value	Changed value	Unit
Maximum growth rate of autotrops	Mua	0.9	0.8	1/d
Maximum growth rate of heterotrophs	Muh	6	5	1/d
Maximum growth rate of poly-p biomass	Mup	1	0.9	1/d
Decay rate of autotrophs	Ba	0.15	0.2	1/d
Decay rate of heterotrophs	Bh	0.62	1	1/d
Saturation for ammonium in heterotrophs	Knh	0.05	0.030	gN/m^3
Saturation for ammonium in autotrophs	Kna	0.2	0.3	gN/m^3
Saturation for nitrate	Knoh	0.5	2	gN/m^3
Autotrophic yield	Ya	0.24	0.245	gCOD/N
Saturation for acetate	Kah	3	4	$gCOD/m^3$
Fraction of phosphorus in inert COD	fpi	0.010	0.060	gP/gCOD
Nitrogen content in biomass	fnb	0.086	0.080	gN/gCOD
Rate for PHA uptake	Qpha	1.8	2	1/d
Conversion from Xpf to SS	Xpf-SS	4.9	5	gSS/gP
Phosphorus content in biomass	fpb	0.015	0.018	gP/gCOD
Fraction of readily degradable COD in particulate COD	fss	0.100	0.120	gCOD/gCOD
Fraction of denitrifiers	etag	0.6	0.4	
Aerobic saturation for poly-p	Kpp2	0.010	0,050	gP/gCOD

Similarly other parameters were also changed by trial and error to bring the final effluent of model close to the effluent from WWTP. The effluent parameter before and calibration is presented in Table B5.

Table B5: Variation of model parameters in the secondary settlers before and after calibration

Concentration	**SS** (mg/l)	**Total N** (mg/l)	**NH4-N** (mg/l)	**NO_x-N** (mg/l)	**Total P** (mg/l)	**Total P(filtrated)** (mg/l)	**Total COD**
Observed effluent from secondary settler at WWTP	13.48	9.67	1.55	7.79	3.62	0.874	Less than 30
Simulated final effluent before calibration	20.65	5.47	0.78	3.50	1.75	0.90	Less than 30
Simulated final effluent after calibration	13.61	9.94	1.50	7.27	1.51	0.88	Less than 30

Appendix C: Article

Troubleshooting for improved bio P at Lundåkraverket wastewater treatment plant, Landskrona, Sweden

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Article Information

Keywords:
Enhanced biological phosphorus removal (EBPR)
Biodenitro
EFOR
WWTP

Abstract

Phosphorus is said to be one of the key nutrients responsible for eutrophication. Lundåkraverket is a modern WWTP situated at the municipality of Landskrona, Sweden. Occasional P peaks have been observed at Lundåkraverket with P concentration as high as 5.1 mg/l. An attempt was made to study these P peaks with the help of computer program EFOR. Only the biological part was modelled and the chemical part was ignored. A constant wastewater flow of 700 m^3/h and yearly average values were taken as inputs for the wastewater composition. A constant temperature of 20°C was used throughout the simulations. Factors affecting bio P process such as diluted wastewater, low-flow conditions, temperature effects, high acetate concentration in wastewater and operational problem related to return sludge pump were studied. Effects of a diluted wastewater composition on the P peaks were noticeable with effluent P concentrations rising as high as 3.2 mg/l..

Introduction

The presence of nutrients, specially phosphorus and nitrogen in wastewater can cause serious problems in the receiving water bodies such as eutrophication, algal blooming in natural water. Therefore it is very important to remove phosphorus before being discharged into the receiving waters. Phosphorus in wastewater can be removed by treating it either chemically or biologically or combination of both, in a wastewater treatment plant Principle sources of phosphorus are mainly phosphate detergents and human wastes (Gillberg *et al.*, 2003).

In Enhanced Biological Removal Process (EBPR), a special type of bacteria, the bio-P bacteria or Phosphate Accumulating Organisms (PAOs) is responsible for taking up and storing large quantities of soluble orthophosphate in the form of insoluble polyphosphate in their cells (Janssen *et al.*, 2002). During the anaerobic phase of wastewater treatment, the PAOs take up carbon sources such as acetates or VFAs present in the wastewater and store them as carbon-rich product such as poly-hydroxy-butyrate (PHB). This energy is obtained mainly by the breaking up of the stored polyphosphates which ultimately releases orthophosphates in the water phase. In the subsequent aerobic/anoxic phase the PAOs use the stored PHB as energy source for P uptake, polyphosphate storage and biomass growth. Therefore, phosphate is removed from the wastewater with the help of excess sludge (Janssen *et al.*, 2002).

The main objective of carrying out this study was to thoroughly understand the EBPR process in Lundåkraverket WWTP, Landskrona, and to carry out a study of the ongoing occasional abnormal phosphorus peaks in the outlet with the help of computer modeling. Computer program EFOR was used as a tool for simulation. Annual average values from the available data has been used in the model while daily load variations and storm water effects have been neglected. Effluent concentrations from the individual treatment units particularly, the primary and secondary settlers were not available for accurate calibrations. Lack of appropriate data of incoming wastewater flow and the effluent parameters from trickling filters are also a constraint that has limited the study.

Computer modelling is strictly limited only to the biological treatment. Chemical treatment, which is also the final treatment in Lundåkraverket WWTP, has been excluded. Therefore, elimination of this process in the modelling may limit the accuracy of the results. As a major portion of the wastewater treatment is based on EBPR, main focus of study is based on biological treatment.

Description of Lundåkraverket WWTP

Lundåkraverket WWTP is situated in the municipality of Landskrona. It serves a population of approximately 37,000 people. The incoming pump station has a capacity of 3050 m^3/hour, and the normal average flow is about 700 m^3/hour.

Incoming wastewater composition

The annual average values of the incoming wastewater composition to Lundåkraverket is shown in Table 1.

Table 1: Incoming wastewater composition to Lundåkraverket (2005-2006)

Parameter	BOD_7	CODt	Tot-P	Tot-N	NH_4-N	SS
Average (mg/l)	109	333	6.0	35.4	23	221

Description of the different units

Wastewater at Lundåkraverket goes through three main treatment processes – mechanical, biological and finally chemical treatments. Mechanical treatment mainly comprises of bar screens, sand trap and/or grit removal and also the primary treatment, In the primary treatment there are two pre-settlers – primary settler 1 and primary settler 2. Biological treatment consists of the anaerobic phase - bio P basin 1 and bio P basin 2, aerobic/anoxic phase with two coupled biodenitro tanks and finally two parallel secondary settlers. The main feature of the biodenitro tank is that it can be operated under both anoxic and aerobic phase depending on the different phases of the operational cycle. Mainly, a four phase cycle is in operation. The anaerobic basins are placed before the biodenitro basins. This type of configuration is patented process biodenipho configuration which is used to enhance the bio P process. The flow sheet is shown in Figure 1.

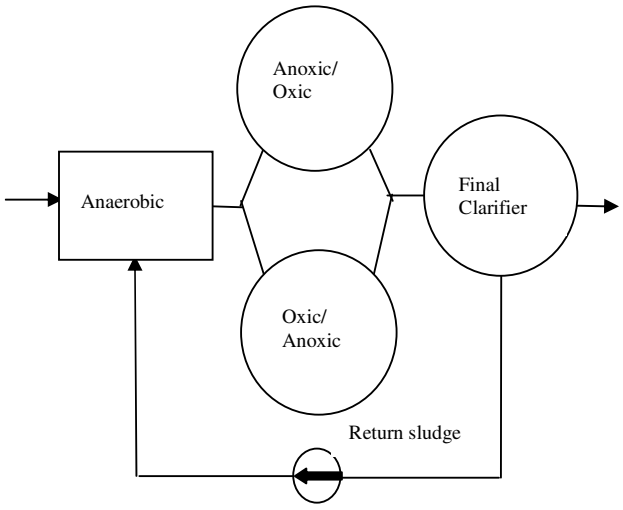

Figure 1: Schematic of biodenipho process

Final treatment at Lundåkraverket consists of chemical treatment that is used to remove additional suspended materials and phosphorus concentrations from the WWTP. They consist of flocculation basin and lamella sedimentation.

Observed P peaks

Occasional phosphorus peaks have been observed in the effluent at Lundåkraverket WWTP. From the available data for periods between 1997 to 2006 it has been observed that the most severe conditions (high P peaks) were recorded between the years 2002 and 2004 with effluent P concentrations as high as 5.1 mg/l. An example is shown in Figure 2. The peaks normally lasted for a week and after that it disappeared. Time interval between these peaks were mostly 3-4 weeks but sometimes it appeared more frequently. Concerned authorities at Lundåkraverket WWTP are trying hard to find a solution to this problem but with little success.

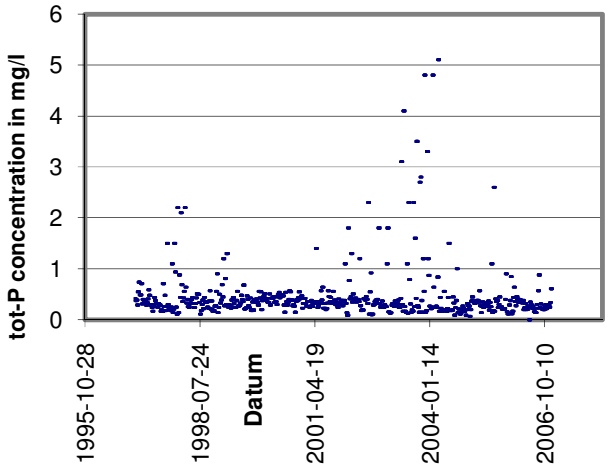

Figure 2: Effluent P concentrations at Lundåkraverket WWTP during 1997-2006

Computer modelling

Computer program EFOR was used as a tool for simulation of the process conditions at Lundåkraverket WWTP. Sludge treatment and chemical treatment has been ignored in the model. The different units of the model is shown in Figure 3. There are two primary settlers - PS1 and PS2. Dosing 1 represents the rejectwater from secondary sludge which is connected to node 1 through pump 3. WS2 represents the waste sludge from the primary settlers PS1 and

PS2 which will undergo sludge treatment and are connected through node 3 and pump 4.

AS3, AS4, AS5 and AS6 represent the anaerobic tanks whereas AS1 and AS2 represent the biodenitro tanks. SS1 represents the secondary settler. The line connecting SS1 to AS3 represents the return sludge and pump 1 is used to control this activity. A major portion (90%) of the wastewater coming out from the primary settler will pass to the tank AS4 and 10% of the flow will pass to tank AS3.

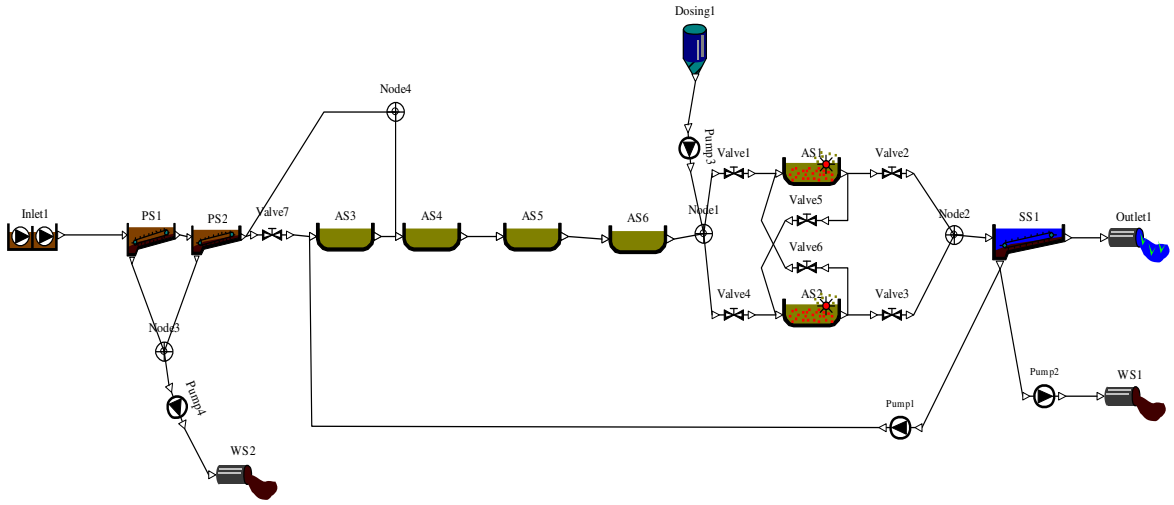

Figure 3: Schematic showing the different units in the model

Results and discussion

In order to find possible explanations regarding the occasional phosphorus peaks in the effluent, a number of process conditions responsible for the deterioration of bioP process were simulated with the help of EFOR. The different conditions that were tried to check the P peaks in the effluent is described below.

Effect of diluted water

High flows during low loaded conditions cause dilution of the incoming wastewater. This can have an adverse effect on the effluent concentrations due to deficiency of available substrates which ultimately impacts the bio P efficiency of the WWTP.The effects of this diluted wastewater periods were simulated and results were plotted. In order to check the effects of a diluted wastewater in the model, the model was run for 24 hours with a 22 hour initial dilution period and the remaining 2 hours with a normal loaded period. The effect of this condition on the phosphorus effluent in the model is represented by simulation day 1 in Figure 4. Similarly, the model was run for 2, 3, 4, 5, 6 and 7 days in order to check the impact of the 22 hour dilution period in the coming 6 days having a normal loaded condition. In the model this kind of dilution period was achieved by increasing the incoming wastewater flow and decreasing the loading of COD, BOD,

phosphorus and acetate concentrations in the influent.

From Figure 4 it can be seen that the phosphorus concentration on the outlet was almost 8 times (3.2 mg/l) higher than the normal effluent values (0.39 mg/l) on the 1st day of the simulation. This peak gradually decreased with time and on the 7th day of simulation the P concentration was 0.5 mg/l.

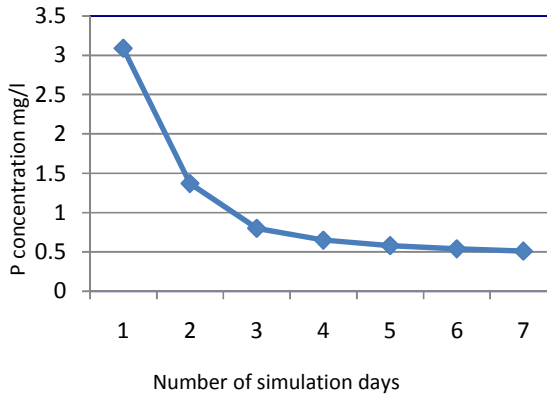

Figure 4: Effect of a 22 hour dilution period on P peak

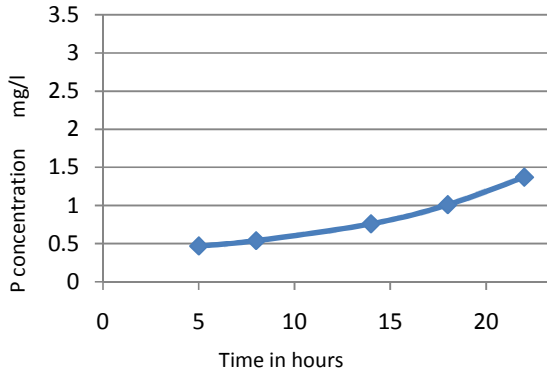

Figure 5: P concentration with 5, 8, 14, 18 and 22 hour's dilution period for a 2 day simulation

In the next part a dilution period of 5, 8, 14, and 22 hours were studied for a 2 days time period. This was done by running the model for an initial 5 hour dilution period and the remaining 42 hours with a normally loaded period The dilution period was subsequently increased from 5 hours to 22 hours and its impact on the effluent P peaks were observed. Figure 5 clearly illustrates that longer the incoming wastewater dilution period, the more will be the P peaks at the outlets. After 2 days simulation, the outlet value of the P peak for a dilution period of 5 hours is 0.47 mg/l whereas for a 22 hours dilution period the P concentration was observed to be 1.37 mg/l.

From Figure 6 it is noticed that with a 2 days simulation, the orthophosphate concentration during the normally loaded condition is being gradually released in the anaerobic tanks AS4 to AS6 and after that there is a P uptake taking place in one of the biodenitro tank AS1. But in the diluted condition the orthophosphate concentration is almost constant. This deterioration of the bio P process leading to a high P concentration at the outlet could be due to lack of enough substrate for denitrification and PHB storage by PAOs. This causes a high nitrate concentration in the outlet as well as in the return sludge which further worsens the bio P efficiency as the anaerobic tanks are not fully anaerobic or is in an anoxic state as shown in Figure 7. The presence of nitrate in the anaerobic tanks prevent P release (orthophosphate release) in the anaerobic tanks. The proportional variation of nitrate in return sludge and phosphorus in the outlet is illustrated in Figure 8.

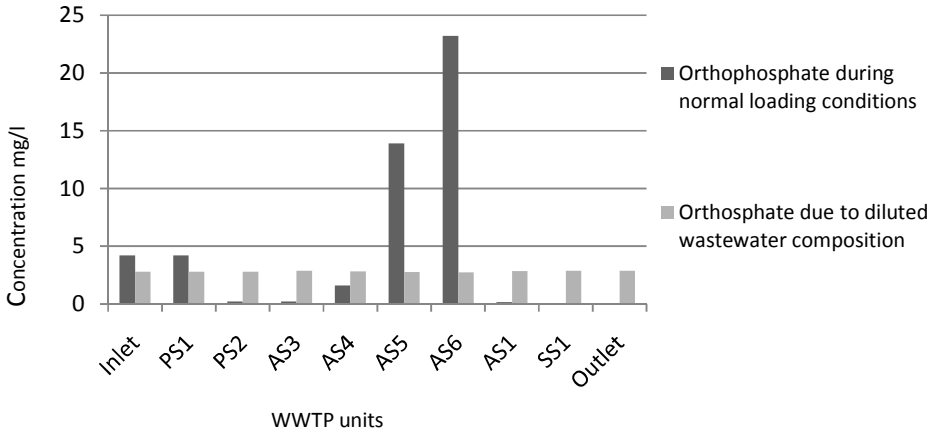

Figure 6: Variation in Orthophosphate concentrations in different units during normally loaded and 22 hours of dilution period with a 2 days simulation

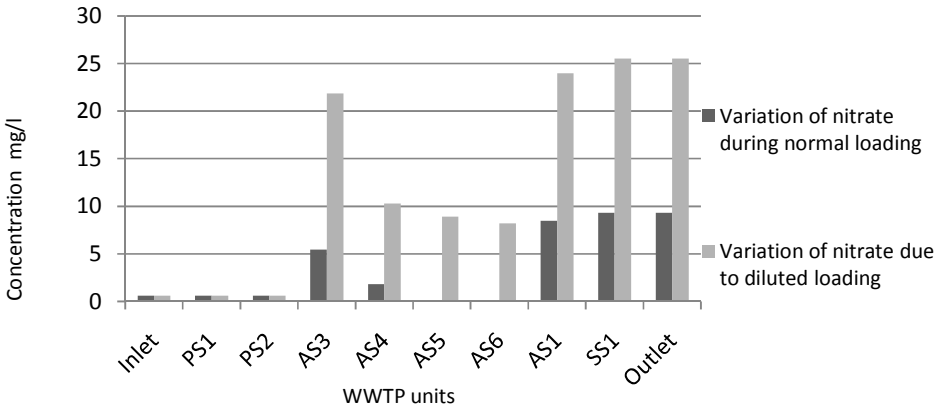

Figure 7: Variation of nitrate concentrations in the WWTP units during normally loaded and 22 hours of dilution period with a 2 days simulation

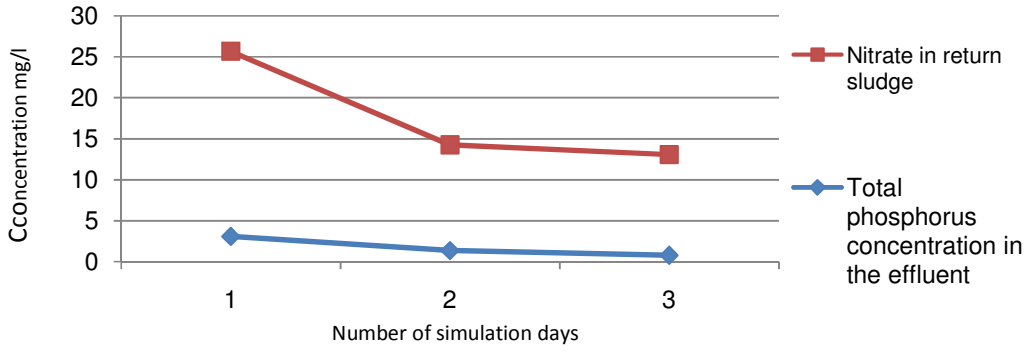

Figure 8: Effect of 22 hours dilution period of incoming wastewater in the effluent P concentration and Nitrate in the return sludge in different simulated days

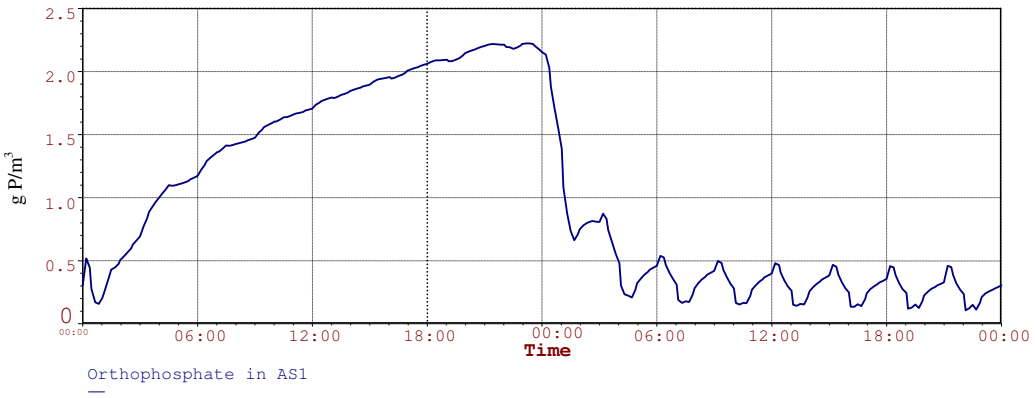

Figure 9: Dynamic variation of orthophosphate in AS1 with 2 days simulations and 22 hours dilution period

Dynamic variations of orthophosphate concentration in one of the biodenitro tanks AS1 during the 2 days simulation for a dilution period of 22 hours is presented Dynamic variations of orthophosphate concentration in one of the biodenitro tanks AS1 during the 2 days simulation for a dilution period of 22 hours is presented in Figure 9. From Figure 9 it can be seen that in AS1, P uptake is taking place only after 24 hours. The absence of a P uptake during the first 24 hours reflects the effect of the dilution period where P uptake is deteriorated. This may be due to the fact that less substrate is available for the bio P bacteria to store it as PHB which influenced the P uptake in the aerobic tank AS1

Effects of low flow condition

The effect of low flow condition were also studied by reducing the incoming flowrate concentration from 700 m^3/hr to 300 m^3/hr for a duration of 5 hours and 10 hours. The result of this change is presented in Table 2.

Table 2: Effect of low flow condition

Duration of low flow condition (hrs)	Simulation days (d)	Total P concentration at the outlet (mg/l)
5	1	0.44
5	2	0.40
10	1	0.44
10	2	0.42

From Table 2 it is observed that the P concentrations have slightly increased with an incoming low loading of 5 and 10 hours but is still within the outlet limit of 0.5 mg/l at Lundåkraverket.

Figure 10 shows the dynamic variation of the sludge blanket in the secondary settler for a one day simulation for a 5 hour initial low-flow condition. It is seen that there is no significant variation in the sludge blanket due to the low-flow condition

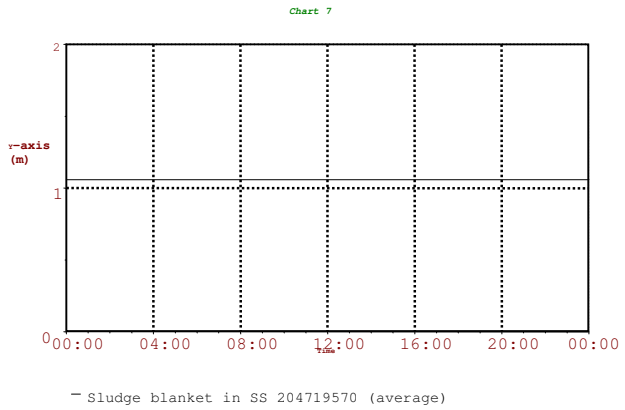

Figure 10: Dynamic variation showing the sludge blanket in the secondary settler for low-flow condition

Temperature effects

In order to check the temperature effects on the P peaks the wastewater temperature was decreased from 20°C to 15°C and 10°C for a three day period. In both the cases, no effect on the P peaks were observed.

High acetate in the wastewater composition

An external dosing with a high acetate dosing was introduced just before the primary settlers to check its effects on the effluent P concentrations. Acetate favours bio P efficiency. But higher concentration in the wastewater influent may sometimes favour the growth of GAOs. The growth of GAOs effect the substrate storage of the PAOs due to competition of substrate by GAOs in the anaerobic section thereby deteriorating the bio P efficiency. It was observed that the P concentration decreased from 0.39 mg/l to 0.17 mg/l. Further increase in the acetate concentration did not show any deterioration of bio P but it enhanced the bio P efficiency perhaps due to the readily available substrate.

Operational problem with the return sludge pump

An operational problem that can rise if the return sludge is stopped few hours was studied. This was achieved in the model by changing the operation of the return sludge pump by introducing a 3 phase cycle instead of the normal one phase cycle. In a 72 hour cycle, the first phase was assigned

a duration of 48 hours, the second phase with a duration 6 hours and the third phase with a duration of 18 hours. The pump was operated with a return sludge flow of 70 % of the incoming flow for the 1st and the 3rd phase. On the other hand in the 2nd phase the set point was set to zero in order to stop the pump for six hours. After running the simulations, the effluent values of phosphorus concentrations did not show any alteration. But while observing one of the aeration tanks AS1 during the dynamic simulations, it was noticed that orthophosphate, SS and nitrate concentrations showed a sharp deviation at that particular time when the pump was turned off. But the curve eventually smoothed off in the next phase. Figure 11 shows a sharp increase in the sludge blanket during the stopping of pump for 6 hours. This sudden change smoothed out with time and was normalised. No P peaks was observed in the effluent due to this phenomena.

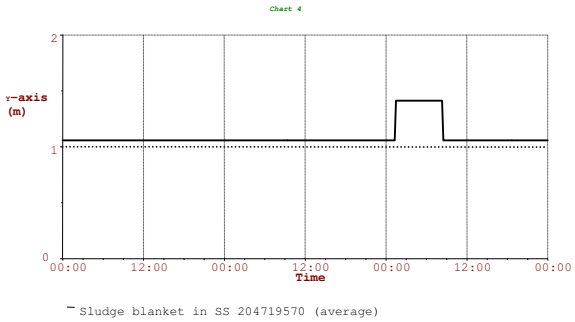

Figure 11: Dynamic variation of sludge blanket in secondary settler after stopping the return sludge pump

Effect of trickling filters

The effect of trickling filters couldn't be modelled in this study due to lack of sufficient information about its effluent parameters. There is a possibility that during high flow conditions when more effluent from the trickling filter is added before the aeration basin, more nitrate and phosphorus is added to the aeration basins. This may worsen the bio P efficiency during the dilution period. This is because the water added from the trickling filters do not posses PAOs that has stored PHB for P uptake. Due to less substrate availability in the aeration basin, more nitrate will be added to anaerobic basin via the return sludge which will affect the PHB storage of the PAOs and subsequently the P uptake will be hindered.

Conclusions

The diluted incoming wastewater with a longer period of time can increase the effluent phosphorus peaks that gradually decreases in about a week. This kind of situation is likely to occur during low flow weekend conditions with the dilution caused by prolonged rainfall.

During the dilution period there will be deficiency of substrate for denitrification which will cause an impact on the anaerobic P release and subsequent aerobic P uptake. P release is deteriorated due to lack of readily degradable substrate and the presence of nitrate in the anaerobic tanks from the return sludge stream.

A low-flow condition as high as 10 hours doesn't show any P peaks. So, there is less probability of occurrence of the P peaks in the effluent at Lundåkraverket WWTP due to low-flow condition.

EFOR is a helpful tool for studying the different process conditions in a WWTP. But the accuracy of the results vary depending upon available data required for proper calibration, amount of time spent on the model as well as a thorough knowledge about the WWTP operations.

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